TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

TRANSPORTATION LIBRARY
UNIVERSITY OF MICHIGAN

VOL. LIII.

DECEMBER, 1904.

Edited by the Secretary, under the direction of the Committee on Publications.

Reprints from this publication, which is copyrighted, may be made on condition that the full title of Paper, name of Author and page reference are given.

NEW YORK:

PUBLISHED BY THE SOCIETY.

1904.

Transportation
Library
TA
1
.A526

Entered according to Act of Congress, by the AMERICAN SOCIETY OF CIVIL ENGINEERS, in the Office of the Librarian of Congress, at Washington.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CONTENTS.

PAPERS.

No.		PAGE.
978	**THE COLLAPSE OF A BUILDING DURING CONSTRUCTION.**	
	By **H. de B. Parsons**	1
	Discussion on Paper No. 978:	
	By NATHANIEL ROBERTS	10
	C. J. TILDEN	13
	F. T. LLEWELLYN	15
	JAMES P. WHISKEMAN	17
	GUY B. WAITE	21
	C. C. SCHNEIDER	24
	OSCAR LOWINSON	26
	H. P. MACDONALD	30
	J. H. O'BRIEN	31
	GEORGE A. JUST	32
	H. DE B. PARSONS	35
979	**THE GATUN DAM.**	
	By **C. D. Ward**	36
	Discussion on Paper No. 979:	
	By CHARLES L. HARRISON	41
980	**ON SEDIMENTATION.**	
	By **Allen Hazen**	45
	Discussion on Paper No. 980:	
	By GALEN W. PEARSONS	72
	ROBERT SPURR WESTON	74
	GEORGE W. FULLER	77
	ALLEN HAZEN	87
981	**LAKE CHEESMAN DAM AND RESERVOIR.**	
	By **Charles L. Harrison and Silas H. Woodard**	89
	Discussion on Paper No. 981:	
	By JOSEPH P. FRIZELL	133
	BURR BASSELL	137
	F. B. MALTBY	138
	E. SHERMAN GOULD	139
	E. W. HARRISON	144
	FRANK C. HORN	146
	CHARLES S. GOWEN	147
	EDWARD WEGMANN	150
	J. WALDO SMITH	152
	E. KUICHLING	153
	R. SHIRREFFS	155
	GEORGE Y. WISNER	170
	EDWIN DURYEA, JR	172
	G. S. WILLIAMS	182
	CHARLES L. HARRISON	202
	SILAS H. WOODARD	205

No.		PAGE.
982	**FILTRATION FOR PUBLIC WATER SUPPLIES, WITH ESPECIAL REFERENCE TO THE DOUBLE FILTRATION PLANT AT BREMEN, GERMANY.**	
	By **Eugen Goetze**	210
	Discussion on Paper No. 982:	
	By W. Kiersted	228
	Howard W. Underwood	229
	William B. Fuller	231
	George A. Soper	232
	T. H. Wiggin	235
	Morris Knowles	238
	Allen Hazen	245
	John H. Gregory	250
	William R. Copeland	254
	L. J. Le Conte	256
	J. P. A. Maignen	257
	George W. Fuller	263
	William B. Bryan	265
	Rudolph Hering	269
983	**LATERAL EARTH PRESSURES AND RELATED PHENOMENA.**	
	By **E. P. Goodrich**	272
	Discussion on Paper No. 983:	
	By M. F. Bonzano	305
	V. H. Hewes	306
	H. F. Dunham	306
	Robert Brewster Stanton	307
	Richard Lamb	314
	E. P. Goodrich	317
984	**A PHENOMENAL LAND SLIDE.**	
	By **D. D. Clarke**	322
	Discussion on Paper No. 984:	
	By George L. Dillman	398
	Arthur L. Adams	400
	James D. Schuyler	405
	D. D. Clarke	409
985	**GENERAL METHODS FOR THE CALCULATION OF STATICALLY INDETERMINATE BRIDGES, AS USED IN THE CHECK CALCULATIONS OF DESIGNS FOR THE MANHATTAN BRIDGE AND THE BLACKWELL'S ISLAND BRIDGE, NEW YORK.**	
	By **Frank H. Cilley**	413
986	**ADDRESS AT THE THIRTY-SIXTH ANNUAL CONVENTION, ST. LOUIS, MO., OCTOBER 3d, 1904.**	
	By **Charles Hermany**	452
987	**SOME NOTES ON THE CREEPING OF RAILS.**	
	By **Samuel Tobias Wagner**	466
	Discussion on Paper No. 987:	
	By Gustav Lindenthal	479
	W. M. Camp	481
	F. S. Stevens	485
	Hunter McDonald	487
	P. H. Dudley	488
	George Tatnall	491
	Joseph S. Ward	493
	Charles Rufus Harte	503
	Samuel Tobias Wagner	505

MEMOIRS OF DECEASED MEMBERS.

	PAGE.
GEORGE CURTIS TINGLEY, M. AM. SOC. C. E.	510

PLATES.

PLATE.		PAPER.	PAGE.
I.	Plan and Profile of the Gatun Dam	979	89
II.	Site of Lake Cheesman Dam	981	93
III.	Views of Lake Cheesman Dam and Reservoir	981	97
IV.	Contour Plan of Lake Cheesman Dam	981	99
V.	Views of Lake Cheesman Dam During Construction	981	101
VI.	Tunnel Entrance and Valves, Lake Cheesman Dam	981	103
VII.	Views of Lake Cheesman Dam During Construction	981	107
VIII.	Lake Cheesman Dam Completed	981	147
IX.	Six-Mile Creek Dam: Site and Construction	981	185
X.	Six-Mile Creek Dam: Preparing Foundation and Up-Stream View	981	187
XI.	Six-Mile Creek Dam: Sections and Completed Dam	981	197
XII.	Filters at Bremen, Germany	982	215
XIII.	Filters and Sand-Washer at Bremen, Germany	982	217
XIV.	Sand-Washer, Pipes, etc., Upper Roxborough Filters, Philadelphia, Pa.	982	231
XV.	Sand-Washing Apparatus, East London Water-Works	982	267
XVI.	Method of Determining the Pressure and Deflection in Sheet Piling	983	281
XVII.	Map of Sliding Land, etc., Portland, Oregon	984	323
XVIII.	Views of Reservoirs and Sliding Land at Portland, Oregon	984	327
XIX.	Views of Cable Railway, Sliding Land, etc., at Portland, Oregon	984	329
XX.	Views of Reservoirs, Portland, Oregon, Showing Cracks in Walls	984	337
XXI.	Views of Reservoirs, Portland, Oregon, Showing Cracks in Buttresses	984	339
XXII.	Map showing Borings and Shafts; also Cross-Sections, Portland Land Slide	984	343
XXIII.	Sections of Borings in Slide, Portland, Oregon	984	345
XXIV.	Sections of Borings and Shafts in Slide, Portland, Oregon	984	349
XXV.	Diagram Comparing Rainfall and Slide Movement, Portland, Oregon	984	381
XXVI.	Profiles of Drainage Tunnels, Portland, Oregon	984	387
XXVII.	Profile of Cable Railway, Portland, Oregon	984	409
XXVIII.	View of Sliding Land, Portland, Oregon; and Clay Taken from Bottom of Shaft	984	411
XXIX.	Fragments of Steel from Locomotive Tires	987	471
XXX.	Movement of Rails: Atlantic City R. R.	987	473
XXXI.	Movement of Rails: New York Branch, P. & R. R. R.	987	475
XXXII.	Movement of Rails: Bethlehem Branch, P. & R. R. R.	987	477
XXXIII.	Movement of Rails: Frackville and Mahanoy & Shamokin Branches, P. & R. R. R.	987	479
XXXIV.	Movement of Rails: New Haven Improvement, N. Y., N. H. & H. R. R.	987	505

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 978.

THE COLLAPSE OF A BUILDING DURING CONSTRUCTION.*

By H. de B. Parsons, M. Am. Soc. C. E.

With Discussion by Messrs. Nathaniel Roberts, C. J. Tilden, F. T. Llewellyn, James P. Whiskeman, Guy B. Waite, C. C. Schneider, Oscar Lowinson, H. P. Macdonald, J. H. O'Brien, George A. Just and H. de B. Parsons.

At the request of some of the members of the Society, the writer presents this brief description of the construction and collapse of the Hotel Darlington, with the object of bringing out a discussion which may lead to improvement in the designs of such buildings.

Construction.—The building was being erected on lots known as Nos. 59 and 59½ West 46th Street, Borough of Manhattan, City of New York. The land measured about 55 ft. in width by 100 ft. 5 ins. in depth. On the east and west sides, the adjacent buildings were of the ordinary construction for residences—of brick with brownstone fronts, four stories and basement in height.

The Hotel Darlington, as the new structure was to be called, was being constructed on the "cage" system, that is, all the weight was supported on the columns.

The outside walls were of brick, and partly concealed the exterior row of columns. These walls were continuous from the foundation, and did not rest on girders at the floor levels.

* Presented at the meeting of May 18th, 1904.

The columns were of cast iron, with end flanges, and with side brackets and lugs cast near the top to carry the girders. All the columns were of rectangular section, generally varying from 9 ins. square by 1 in. thick, in the basement, to 6 ins. square by $\frac{3}{4}$ in. thick on the upper floors. Each column had top and bottom flanges cast only on the north and south sides, and each flange had two bolts. Fig. 1 illustrates a typical column. The thickness varied with the column position in the building. Some of the 9-in. columns in the basement were $\frac{3}{4}$ in. in thickness. Fig. 2 illustrates a typical column foundation.

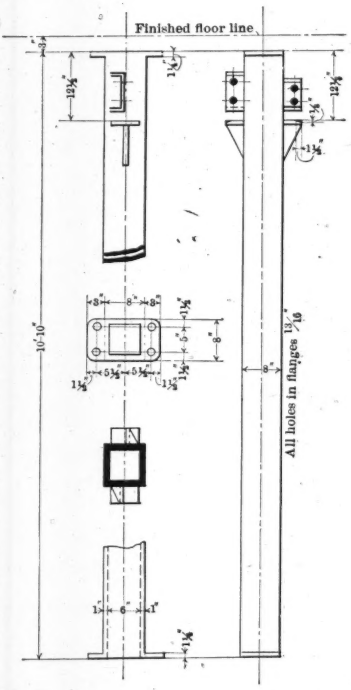


FIG. 1.

The girders and floor beams were of steel. The girders, generally, ran east and west across the building. They were supported on the brackets cast on the columns, and were bolted to the lugs with two bolts at each end. The beams, generally, ran north and south, and were bolted to the girders by double-clip angles with two or three bolts through each leg of the angles. The floor members were all of rolled sections—I's and channels—and many were marked "Phœnix and "Carnegie." Fig. 3 illustrates a typical floor plan and shows the location of the columns.

The floor arches were of cinder concrete, laid up on the Roebling system. The spans were variable, averaging from 4 ft. 6 ins. to more than 5 ft. The thickness of the arches could not be determined, as all of them were destroyed.

The building was to have been twelve stories and a basement in

height, with a penthouse, or one-story structure, partly covering the roof. Each floor was lettered, and the columns on the same floor varied in size. Table 1 is a specimen schedule.

TABLE 1.

Floor.	Letter.	Column.		Height.
		Side.	Thickness.	
Basement	*A*	9 ins.	1 in.	10 ft. 6 ins.
Ground Floor	*B*	9 "	¾ "	14 " 6 "
2d "	*C*	8 "	¾ "	10 " 10 "
3d "	*D*	8 "	¾ "	10 " 10 "
4th "	*E*	7 "	¾ "	10 " 10 "
5th "	*F*	7 "	¾ "	10 " 10 "
6th "	*G*	6 "	¾ "	10 " 10 "
7th "	*H*	6 "	¾ "	10 " 10 "
8th "	*J*	6 "	¾ "	10 " 10 "
9th "	*K*	6 "	¾ "	10 " 10 "
10th "	*L*	6 "	¾ "	10 " 10 "
11th "	*M*	6 "	¾ "	10 " 10 "
12th "	*N*	6 "	¾ "	10 " 10 "
Penthouse				
Height to under side of roof beams				144 ft. 2 ins.

The construction had progressed until the metal work for the ninth floor had been erected complete, and some of the work for the tenth and eleventh floors was in place. The structure collapsed at about 1.30 P. M., on Wednesday, March 2d, 1904.

Observations on the Wreckage.—On collapsing, the building fell downward with a leaning to the north, that is, toward the rear. As viewed from a sixth-story window in the Hotel Patterson, situated immediately in the rear, on 47th Street, the "center of fall," toward which all the various members pointed, more or less, was a little to the west of the center line of the building and a little to the north of a line drawn between the rear walls of the adjacent buildings, Nos. 57 and 61 West 46th Street. Practically all the material fell inward, except a relatively small amount, which fell over the rear wall against and into the Hotel Patterson.

The columns in the outer walls broke off, with approximate uniformity, between the second and third floors, although the floor systems of the stories below, and the interior columns (except some in the basement) were carried away, leaving the outer columns standing, from their foundations to the points of rupture. See Fig. 4.

The columns were measured with calipers and showed a fair uniformity as to thickness of metal on opposite sides. The variation was about $\frac{1}{8}$ in. or less.

In the great majority of cases the columns broke at the flanges, leaving one or both of the broken flanges bolted to the corresponding

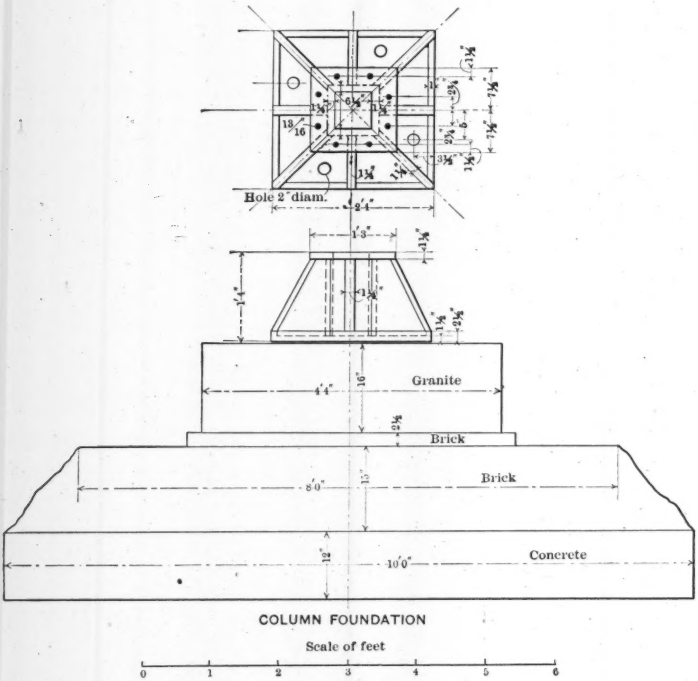


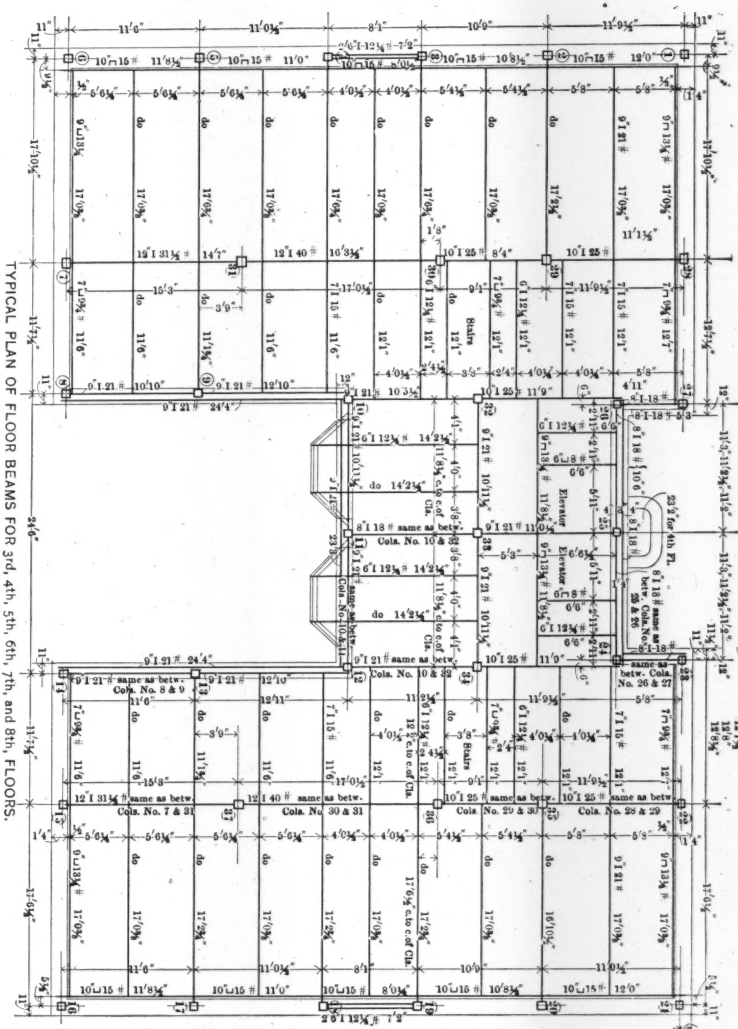
FIG. 2.

flange of the adjacent story column. A few of the columns were broken at or near the center.

There were no ribs cast on, to stiffen the flanges.

The column ends were faced. No shims were found between the ends of the columns.

The holes in the column flanges were drilled, $\frac{13}{16}$ and $\frac{7}{8}$ in. in diameter. The bolts were $\frac{3}{4}$ in., and some were loose. The holes in the



TYPICAL PLAN OF FLOOR BEAMS FOR 3rd, 4th, 5th, 6th, 7th, and 8th, FLOORS.

WITH DIMENSIONS FOR THE FOURTH FLOOR.

BUILDING ON NORTH SIDE OF 46TH ST., 205 FT. EAST OF 8TH AVE.

FIG. 3.

lugs for the girders were cast, and were about $\frac{7}{8}$ in. in diameter. The holes in the girders were about $\frac{7}{8}$ in. in diameter. The bolts were $\frac{3}{4}$-in.

The fractures in the body of the cast-iron columns exhibited fairly good metal. The fractures at the flanges exhibited blow-holes and honey-combing in many instances. About 15% of the flanges were honey-combed, about 25% were defective, and about 60% were fair to good. The defects did not show on the exterior surfaces. The flanges varied from 1 in. to $1\frac{1}{4}$ ins. in thickness.

The rolled-steel members of the floor systems were twisted. The steel gave evidence of being good material, and the bolts held well. A number of pieces—L's and channels—were taken from the débris, which, from the appearance of the bolt-holes, had never been erected. Such pieces were marked for use on the 10th, 11th and 12th floors. They were found chiefly on top of the ruins and near the rear. On March 6th about twenty-two unerected pieces were taken out during the morning. The weight of this unerected material was between two and three tons. Also, a broken piece of an 11th-story column (*M* 32), which had been erected, was taken out with them.

The concrete from the floor arches was broken, and few lumps larger than 12 by 12 ins. were found.

The foundations under the exterior and interior columns were found true and level. The cast-iron shoes under the columns were in place and unbroken. The soil under the column foundations was clay in every instance where dug up, both at the front and back of the lot. The foundations had an area of about 10 ft. square, or amply sufficient to have sustained the load intended.

Three hoisting engines and boilers were dug out of the débris comparatively slightly injured. Their boilers were intact.

All members were bolted. Rivets were not used. There were no diagonal braces, corner braces, or any special means used to provide lateral stiffness against wind or side pressure. The columns were held in their vertical positions by the floor systems resting on the bearing brackets.

Cause of Fall.—The bolts fastening the girders and beams against the column lugs were of smaller diameter than the holes, so that the columns received little or no lateral support. The column lengths were bolted together at top and bottom, and acted as continuous col-

umns. All the loads were eccentrically supported on the side brackets. The columns were too long to carry the superimposed weight. One column situated at or near the "center of fall" (previously described) broke. The upper part of this column, being deprived of its support,

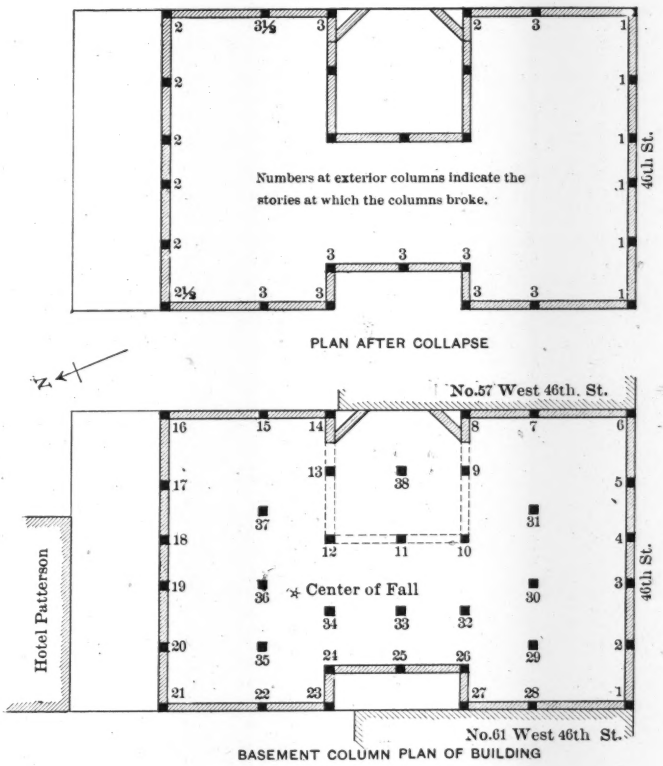
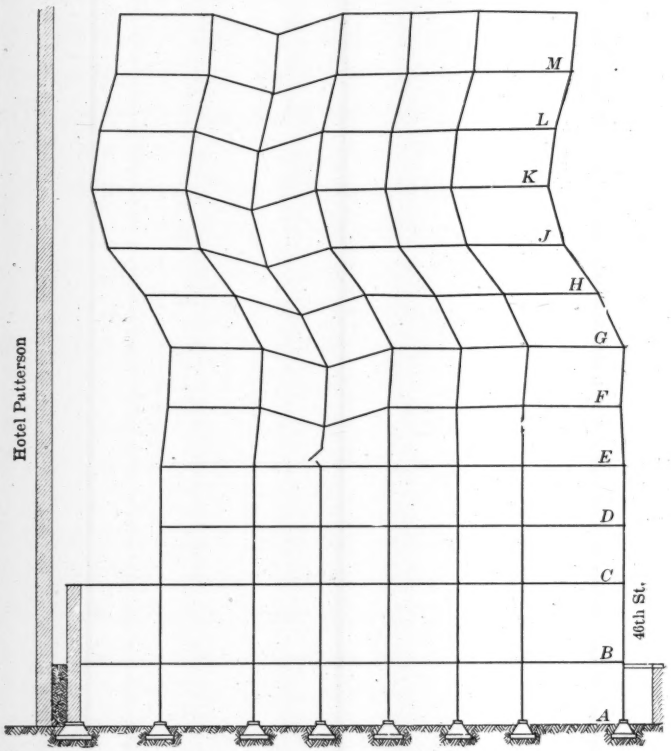


FIG. 4.

fell and pulled with it the floor members bolted to it. Each of these floor members pulled over the adjacent columns to which the other ends were attached; and these columns, having no lateral support, broke at the lower flange, as the pull had a lever arm of about 10 ft., or the length of the column. The above-described action and reaction

of the stresses affected only those portions of the structure above the level of the original fracture.

In falling, the mass of material from above crushed down and broke that part of the structure below the level of the original fracture.



VERTICAL SECTION ILLUSTRATING METHOD OF COLLAPSE
FIG. 5.

The exterior columns did not break off as low down as the interior columns, because the mass fell away and did not crush them as it crushed the interior ones. The uniformity in height at which the exterior columns broke indicates strongly that the primary fracture occurred at or about the level of the fourth floor.

As proof that the top fell into and toward the "center of fall, before the lower part of the structure collapsed, columns *G* 11 and *K* 11 were taken out of the débris near the "center of fall" and from beneath other members which originally were connected at points lower down in the building. Near the same spot, column *D* 36 was found standing in a nearly upright position. Of these columns, the first was broken at the center, and the other two at the flanges.

Furthermore, as more of the structure was south of the "center of fall" than north of it, the northern or rear portion was forced outward against the Hotel Patterson. This action is illustrated in Fig. 5.

As proof that the original break did not occur on the uppermost story, the unerected material and the derrick mast and boom were found on top of the mass of débris.

The structure collapsed because of lack of lateral support for the story columns, permitting them to act as continuous columns having a ratio of length to least radius of gyration in excess of the known limits for safety.

DISCUSSION.

Mr. Roberts. NATHANIEL ROBERTS, M. AM. SOC. C. E. (by letter).—Mr. Parsons' paper should lead to some very interesting discussions on building construction in general and the use of cast-iron columns and details for high buildings, and be productive of good to engineers and the public at large.

When iron construction of buildings was in its infancy, and buildings, usually, did not exceed six stories in height, and were very seldom of fire-proof construction, cast-iron columns were permissible. Their cost, compared with riveted columns, was greatly in their favor; also, the facility with which they were obtained and their ability to withstand fire without collapse, added to the large factor of safety in use twenty-five years ago (a factor of safety of 8 instead of 6, as at the present time), gave a safe construction.

In the early Seventies, the writer designed a large woolen mill in the West of England, the roof and floors of "slow-burning" construction, supported by cast-iron columns. After being in operation for several years, and the floors being saturated with oil, the mill caught fire in the upper story and burned downward like a torch. The 3-in. flooring was entirely destroyed, the 6 by 12-in. joists and the 10 by 16-in. oak girders were charred through, the several lines of heavy wrought-iron shafting were twisted and bent into various serpentine shapes, but the cast-iron columns, although subjected to a prolonged white heat and to streams of water from the fire engines, which caused most of them to crack from top to bottom, remained in place and prevented the collapse of the building, thereby saving the lives of scores of men who were engaged in removing goods from the lower floors.

The New York City building law requires that all wall columns, whether of steel or cast iron, shall be protected by

"A casing of brickwork not less than 8 ins. in thickness on the outer surfaces, nor less than 4 ins. in thickness on the inner surfaces, and all bonded into the brickwork of the enclosure walls."

Also, all interior columns,

"Used to support any fire-proof floor, shall be protected with not less than 2 ins. of fire-proof material, securely applied," but, "the extreme outer edge of lugs, brackets and similar supporting metal may project to within $\frac{7}{8}$ in. of the surface of the fire-proofing."

It is a well-known fact, however, that many of the so-called fire-proofing coverings in use are fire-proof in name only, as was shown by collapsed steel columns in the fires at Pittsburg and Baltimore.

The writer is not an advocate of the use of cast-iron columns in "tall" buildings, or even in buildings of moderate height, if sub-

jected to bending strains caused by wind or eccentric loading, but believes that no one will dispute the fact that cast iron columns of tough, gray iron, free from injurious cold-shuts or blow-holes, will withstand the combined action of fire and water without collapse, while steel columns will not. Mr. Roberts.

The writer has no desire to criticise this paper, but wishes the author had gone into detail a little more, as it seems to be too general.

Mr. Parsons states that the building was constructed on the "cage" system, that is, all the weight was supported on the columns. Then he says, the "walls were continuous from the foundation, and did not rest on girders at the floor levels." In Fig. 3, however, twelve bays have wall girders for "skeleton construction," while the other outside bays are shown as being of "curtain-wall construction."

Mr. Parsons fails to give any data respecting the floor loads used in designing, and, although he gives a specimen column schedule in Table 1, he does not locate the position of the column in the building, to enable anyone to check the sections; and the detail of the 8-in. column, shown in Fig. 1, does not refer to one of the columns given in the specimen schedule, but appears to be either Columns 29, 30, 35 or 36, at about the 4th story.

He also states: "All the loads were eccentrically supported on the side brackets," while a reference to the plan, Fig. 3, shows that the interior columns and quite a few of the wall columns have very little eccentric loading.

He also states that the holes in the girders were about $\frac{7}{8}$ in. in diameter, and that the bolts were $\frac{3}{4}$ in. It is to be regretted that the exact size of the holes was not given. The standard size for punched holes for $\frac{3}{4}$-in. bolts is $\frac{13}{16}$ in. on the punch side and $\frac{7}{8}$ in. on the die side, although some contractors, to facilitate erection and cover careless workmanship, use a $\frac{7}{8}$-in. punch for $\frac{3}{4}$-in. bolts, *viz.*, $\frac{15}{16}$ in. on the punch side and 1 in. on the die side.

By reference to Fig. 3, it will be seen that Columns 31 and 37 are stayed in one direction only, the nearest beam in the other direction being at a distance of 1 ft. $9\frac{1}{2}$ ins. from the center of the column.

Fig. 3 is a typical plan of floor beams for the 3d to the 8th floors, inclusive. *Engineering News* of March 10th, 1904, gave a plan of the same floors, and stated that "the 9th to 12th floors and the penthouse floor are similar in arrangement to that shown, but their framing is somewhat lighter," and "the 1st and 2d floors, due to the addition of framing for the light-courts, are different in arrangement from the tiers above, and the 1st floor is heavier." The floor loads are also given as 60 lbs. live load and 60 lbs. dead load per square foot.

The floor construction is specified as the Roebling, flat, reinforced-concrete type (System B, Style 4), the fire-proof partitions being carried by the floor system.

Mr. Roberts. The thickness of the floors is given as 13 ins. from finished ceiling to finished floor surface. This construction will weigh not less than 65 lbs. per square foot in addition to the weight of the fire-proof partitions; but the floor beams shown have been designed for a gross load of 120 lbs. per square foot of floor.

Engineering News also gives a schedule of sizes for Columns 30 and 31, and loads on Column 31, but it is to be regretted that enough data are not given to check some of the wall columns under eccentric loads, say Column 2, which is subjected to a bending moment at each tier due to an eccentric load of 11 400 lbs. To withstand this bending moment requires an increase in metal area in the upper columns equivalent to an added load of about 36 000 lbs. in addition to the actual load itself; while Column 1, with an eccentric load at each tier of only 5 600 lbs., requires an increase in metal area in the upper columns equivalent to an added load of about 32 000 lbs. in addition to the actual load itself.

In regard to the failure of the structure, the writer believes there were several contributing circumstances, *viz.:*

1st.—The columns should have been designed for bending due to eccentric loading, necessitating steel columns, as no dependence can be placed upon a column to resist bending when it may be full of concealed blow-holes.

2d.—The absence of bracing for wind strains, and the torsion on the structure caused by derricks, during the hoisting of material.

3d.—The overloading of the upper stories by building material may have had some effect; but, from information thus far obtainable, the writer believes that torsional strains produced by derricks caused the collapse.

Only a few months ago the writer had his attention called to a building in course of erection in New York City, where steel columns in two-story lengths were used, the erector had the guy-lines for the derricks secured to the tops of the columns at least 10 ft. above any floor beams erected, and the erector was surprised (he ought not to have been) to find that some of the columns had a decided twist in them, and in some cases were more than 5 ins. out of plumb.

During the past fourteen years the writer has designed more than thirty steel buildings in New York City, ranging from twelve to thirty stories in height, besides many smaller buildings, and has found that, although there are some architects who realize the necessity of employing engineers to assist them in designing any but the simplest of steel constructions, the great majority of architects will, with the aid of a rolling-mill handbook and the free services of a contractor's "iron-man," concoct a "design" that Providence sometimes allows to be erected without accident.

In conclusion: Mr. Parsons states that some of the beams and

channels were marked "Phœnix" and some "Carnegie." Why were not some "Pencoyd" beams used, also? For is it not written, "a three-fold cord is not easily broken." Mr. Roberts.

C. J. TILDEN, ASSOC. M. AM. SOC. C. E. (by letter).—The writer is strongly opposed to the use of cast iron as part of the framework in skeleton building construction. The principal objections are its general unreliability as compared with steel, and the apparent impossibility of designing satisfactory details for beam and girder connections, column joints, etc. The Hotel Darlington disaster, however, certainly does not prove anything against cast iron, *per se*, as a structural material; indeed, the columns seem to have acted remarkably well under the severe conditions imposed upon them from lack of lateral support and general bad design. Mr. Tilden.

The author makes note of the fact that each column length had top and bottom flanges cast only on the north and south sides, thereby giving the column no stiffness in an east and west direction, except that derived from the floor girders framing into the column near the top. The value of this stiffening may be seen by reference to Fig. 6. At *A* is shown one length of column, drawn to scale, and at *B* is an enlarged detail of the lug to which the floor girder was bolted. (This sketch applies to the columns numbered 7 or 15 in Fig. 3, showing a typical plan of the floor framing.) The bolts used were $\frac{3}{4}$ in. in diameter, and the holes in the lug and girder web were $\frac{7}{8}$ in. Assuming, when the column is vertical, that the bolts are concentric with the bolt holes, as shown by the heavy lines of Sketch *B*, Fig. 6, a horizontal thrust against the end of the girder would tilt the whole column east or west until the axis of the bolt holes, shown at x x in normal position, assumed the position, $x'x'$, the bolts in this position bearing against the bolt holes on opposite sides, as at *M M*. A movement of this kind and amount would be resisted only by the friction due to

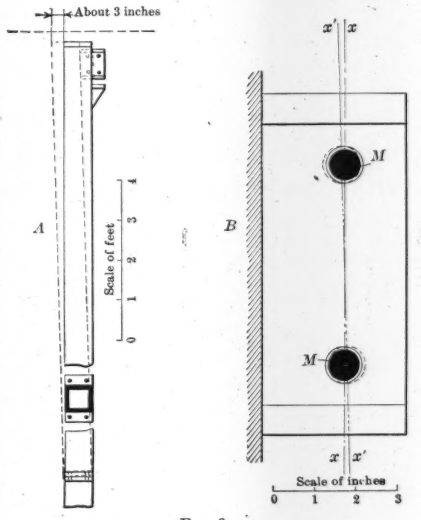


FIG. 6.

Mr. Tilden

tightly set-up bolts, and it is admitted that in most cases these were loose. The effect of such motion on the column is shown in Sketch *A*, Fig. 6. In one story length the column could readily be moved 3 ins. out of plumb. Such a condition of affairs could hardly have existed if steel instead of cast-iron columns had been used in the framing.

Referring again to Fig. 3, it is seen that the foregoing reasoning would apply to the two rows of columns numbered 7, 31, 30, 29, 28; and 15, 37, 36, 35 and 22. That is, practically the whole framework could be tilted east or west until it was out of plumb by an amount equal to 3 ins. for each story length. As a matter of fact, it was brought out in the testimony before the coroner's jury that, a week or two before the collapse, the structure (then about seven or eight stories high) was out of plumb by an amount variously estimated at from 6 ins. to 2 ft. This error was corrected by rope tackle used to jack the leaning framework back into place. The direction of this inclination is not mentioned in the press reports, so that it cannot be stated definitely whether or not the trouble was due directly to the cause just cited. The diagram shows rather strikingly, however, the utter flimsiness of the whole structure, and it is easy to credit the further testimony of witnesses before the coroner that on various occasions the building swayed noticeably.* It is perhaps hardly necessary to speak of the severe shearing stress to which the bolts would be subjected by any distortion of the nature and amount indicated.

One direct violation of the building law may be noted. The statute† reads:

"The top and bottom flanges, seats and lugs (of cast-iron columns) shall be of ample strength, reinforced by fillets and brackets; they shall be not less than 1 in in thickness when finished."

The flanges of columns in the Darlington were not reinforced in any way, and the fact that nearly every column broke in the flanges indicates how serious a defect this was. At the inquest the foreman of the iron workers testified that the Building Department had ordered him on several different occasions to stop work on the iron frame, and that in each case the owner had told him to go ahead regardless of these orders.‡

The real trouble, therefore, would seem to be the lack of a sufficiently severe penalty for violation of the building code, and the subject would seem to be one for discussion by law makers rather than engineers. In relation to this, it may not be amiss to mention the recent discovery by an archæologist of the building laws which were in force during the reign of Hammurabi, King of Babylon, about

* *Engineering News*, March 24th, 1904, p. 281.
† Building Code, Part XXII, Section 112.
‡ *Id. loc. cit.*

2250 B. C. The records* of this wise and just monarch have been deciphered, and, among other ordinances, the one numbered CCXXIX reads: Mr. Tilden.

"If a builder build a house and has not made his work strong, and the house has fallen in and killed the owner of the house, then that builder shall be put to death."

Further ordinances provide that the builder's son shall suffer death, if through his father's negligence the son of the householder is killed, and so on, in strict conformity with the Mosaic doctrine of "an eye for an eye," through the various members of the household. This bit of legal history is respectfully submitted to the law makers of Greater New York as worthy of the careful and reverent consideration to which more than four thousand years of "precedent" would seem to entitle it.

F. T. LLEWELLYN, ASSOC. M. AM. SOC. C. E.—Upon reading Mr. Parsons' description of the collapse of the Darlington Building, and recalling the other detailed articles which have appeared in the technical journals, it is evident that this disaster was caused by the inefficiency of the columns, as stated in the author's concluding paragraph; and it is of interest to inquire if the Darlington was an isolated case of this kind. The writer has looked up all the kindred instances of similar disasters of which any record was available, and gives a short summary thereof, with his sources of information, so as to see what common factor, if any, there may be. Mr. Llewellyn.

The earliest reference found is in an address† given by C. T. Purdy, M. Am. Soc. C. E., before the Boston Society of Civil Engineers, wherein he says:

"The days in which cast-iron columns will be used in the construction of high buildings are fast being numbered. It would hardly seem necessary to compare them with steel columns before an audience of New England men, if we will but recall the Pemberton mill disaster with its frightful loss of life."

The writer has not been able to secure any data regarding this disaster, and would be glad to learn the details.

In *Engineering News* of May 24th, 1894, is described the distortion by wind of the nearly constructed building at No. 14 Maiden Lane, New York City, due to the loosely bolted connections of the cast-iron columns, although in this case they were much better than usual, and the framework was subsequently pulled back.

In *Engineering News* of December 6th, 1894, is given a short account of the failure of the five-story building, belonging to the Montreal Street Railway, which also had cast columns, and which, apparently, gave way after completion by the crushing of a wall.

* Editorial, *American Architect and Building News*, March 26th, 1904.

† *Journal of the Association of Engineering Societies*, March, 1895.

Mr. Llewellyn. Regarding the collapse of the nearly finished Ireland Building on West Broadway and Third Street, New York City, *Engineering News* of August 15th, 1895, says: "The prime cause of the accident appears to us to have been the weakness of the cast-iron columns." In a later issue, August 29th, additional evidence seemed to show that faulty foundations were the primary cause.

Commenting on the failure of the Brown Soap Factory building, on Twelfth Avenue, between Fifty-first and Fifty-second Streets, New York City, *The Engineering Record* of June 12th, 1897, after criticising in detail the cast-column construction, says: "Cast-iron columns are a source of great danger wherever they are used, as we have frequently shown, and no safety factor can remove that danger." And *Engineering News* of June 17th, 1897, concludes an exhaustive review of the same structure with the following:

"Since the failure of the Ashtabula Bridge, twenty years ago, no bridge engineer has dared to risk the safety of a bridge upon the lug of a cast-iron column. Will it require a disaster of similar extent to awaken engineers and architects to their fatuity in continuing the use of such cast-iron columns with all their hidden defects and their eccentrically loaded lugs in buildings the failure of which may cause the loss of hundreds of lives?"

The collapse of the Darlington Building, in which the columns were cast iron, is added to complete the list.

It is noted, of course, in each of the cases cited that cast-iron columns were used in every instance. After a search through all the accessible records, the writer has been unable to find any case of failure through the use of steel columns, and if such exists he would appreciate a description of the details. Although some of these failures were probably induced by other causes, many of them seem to have resulted directly from defective cast columns, and in all cases the collapse was intensified by their lack of stiffness. It may be of interest, therefore, to summarize briefly the properties of cast iron as used in the columns of high buildings, passing over the uncertainty regarding hidden internal defects in their pouring and cooling as sufficiently well known, and assuming their manufacture and details to be up to the standard.

1.—It is impracticable to connect a cast-iron column with the ones immediately above or below it by any kind of splice joint which will preserve their continuity, as, on account of the danger of cracking the castings by riveting, the flanges must be bolted, and, almost invariably, the bolts are a loose fit. Also, the weakness under tension of the cast flanges (which it is very hard to pour sound) makes them a poor medium to transmit the pull of the bolts. Hence a stack of cast columns is little more than a lot of those toy wooden pillars we used to pile end on end when we were children.

2.—This instability could be decreased if each story column were braced by a rigid connection with the horizontal girders and beams,

but, again, the impracticability of riveting and the tensile weakness of the cast lugs result in a loose joint, which leaves the effective unbraced length of the columns the full height of the building instead of story lengths, and throws out the usual formulas of strength. Mr. Llewellyn.

3.—The best specifications require the projecting brackets to be given a slight slope downward, in order to throw the load of the supported beams as close as possible to the column and relieve the tension on the cast shelves, but, in ordinary foundry practice, the accuracy of this slope cannot be ensured. Neither does it seem desirable to reinforce the column at the brackets by an internal diaphragm, for, apart from the additional expense in the moulding, there would arise such uneven cooling stresses as would weaken, rather than strengthen, that place.

In a word, cast iron has proved itself to be a dangerous material for use in the columns of high buildings, and the Darlington disaster, as reported by Mr. Parsons, is merely one more object lesson. Can we afford to look with unconcern upon such construction for those members upon which depend the entire strength of our buildings and the safety of much life and property? All railroad engineers have long since vetoed the use of cast iron for bridges, and, neglecting the commercial side, united effort should be made to bar this material from use as columns in any but low and simple buildings. It is understood that, among other things, the prudence exhibited in the design of railroad bridges is recognized by the insurance companies, who place a premium upon the security of railroad travel by paying twice the amount of their ordinary policies in case of accidents while traveling on the railways. The same protection, of course, would be best afforded by the employment of only competent engineers to cooperate with architects in the design of tall or hazardous buildings, but, as this desideratum cannot always be controlled, the building codes of our great cities should be amended so as to make impossible the flimsy structures now only too common. As stated in a recent editorial on this subject, with steel columns a wholly satisfactory design is possible in all cases, while the limitations of cast iron are firmly fixed.

No reference has been made to the action of cast iron in fires, nor to its durability against corrosion, nor to its actual strength, these matters being foreign to the paper, but it is suggested that the whole question of the behavior of cast iron, steel and other columns, under the various possible conditions, would form a most interesting and pregnant subject for another discussion. The writer would commend this to the notice of the proper committee.

JAMES P. WHISKEMAN, ASSOC. M. AM. SOC. C. E.—The speaker was detailed to make a report on the collapse of the so-called Darlington Hotel, and was on the premises until the wreckage was all cleared Mr. Whiskeman.

Mr. Whiskeman.

away. Without going into details, which are covered by Mr. Parsons' paper, the following can be added to what has already been stated.

The details for the framing were defective in the following respects: Not all the columns were eccentrically loaded, but the majority of them were. The worst cases were in the wall columns. The seats for the channels nearest the walls (which were also girders) projected some 9½ ins. from the face of the column. To this column the channels were bolted with one ⅞-in. bolt about 8 ins. from the face of the column, and with no bracket connection. The plans showed these channels framing into a 9-in. I-beam, which was to be framed close to the column, on proper seats and with a standard bracket. In curtain-wall construction, eccentric loading is quite common, as it is desirable to have the columns embedded in the walls without offsets or projections on the interior of the walls. The columns supporting the walls were eccentrically loaded, but the girders remained level and the columns plumb after the fall.

The column flanges should have extended around the four sides of the columns, instead of on two sides only. This, together with proper bolting, would have given additional strength and rigidity to the framing.

The interior columns on the front and rear were not braced properly in a longitudinal direction. The beams, instead of framing into the columns, were framed to the girders on each side of the columns, and, therefore, were not properly supported in this direction at every floor. The majority of the columns broke at the flanges. The bolting appeared to be loose, and, as the threads were not stripped, it would appear that the columns were not properly bolted together. The bolts on one side were probably loosened in plumbing, and, as no shims were used, the load was concentrated more or less on the opposite flange. The metal in the flanges was honey-combed and full of impurities.

There was also loose and insufficient bolting in the beam and girder framing. In one case three bolts were omitted from a possible four; in another case two bolts out of a possible four. But even where these bolts were omitted the parts remained framed together. An 8-in. I-beam was found in the wreckage bolted at one end to part of a 7 by 7 by 1-in. column with one bolt, and to a lug of a column at the other end with one bolt. This beam had two holes at each end close to the bolt holes, and was weak, but the connection was stronger than the columns.

The cinder concrete of the floor arches was disintegrated in the fall, but this was due to the fact that it had not set properly. The centering had not been removed. Several specimens of this concrete preserved in a warm room became very hard in a few days.

As to the amount of material stored on the floors of the building,

owing to frequent complaints from the neighbors, material was removed from the street as soon as possible after its delivery, and stored in the building. On the first floor, back of the light-shaft and against the west wall was piled a lot of Rosendale cement in bags and barrels, producing a live load of 250 lbs. per square foot on this floor, which was calculated to stand safely 60 lbs.; but the floor did not fail on this account. On the upper floor was distributed, ready for erection, or in piles, considerable iron and steel. In the neighborhood of Column 37, about twenty pieces of unerected iron were found in the wreckage. About 30 tons of unerected material were found in the ruins. Mr. Whiskeman.

The derrick used for hoisting iron was supported on the top of the framework, and in the center of the building near the front. The derrick had a boom probably 60 ft. long. How well the weight of the derrick was distributed, or the framework temporarily reinforced, it is hard to say, but it can be surmised that it was not properly done. It was also probably guyed to the structure in such a way as to strain it excessively. The derrick was not in use on the day of the collapse. There were frequent complaints from the adjoining tenants that the hoisting of materials in the elevators caused their buildings to vibrate.

The building was out of plumb at various times from 6 to 18 ins., and was never properly sway-braced before or after it was pulled back. Excessive flange strains were produced in this way, which could cause the building to collapse at a later date. Columns 36 and 37 were not braced properly longitudinally, and should be treated as long columns, and one of these broke about half-way up, or in the 5th story, and fell. On account of the lack of rigidity, the entire mass was precipitated toward these columns. Columns 34, 36 and 37 were the only ones broken off in the basement. None of the foundations failed in the slightest degree.

The overloading of the floors, by the iron or by the derrick, the loose bolting of all members, the lack of longitudinal supports for the columns, the building being out of plumb and never properly sway-braced, all combined to cause the complete collapse, which any one of them was sufficient to induce.

The structural engineer has learned nothing new from this collapse, as some or all of the defects noted can be found in other buildings of the same class.

The designing of buildings of this class is considered in much too light a vein, and, as the operation is more or less a speculation, the temptation exists to make the structure too light at the expense of rigidity and stiffness. On account of the height being less than four times the width, no permanent wind bracing is provided and as little temporary bracing as can conveniently be put in. Reliance is placed on the curtain walls and fire-proof floors to stiffen the structure, but,

Mr. Whiskeman.

in reality, the framework is sometimes nearly completed before the walls are begun, therefore elements of strength are depended upon which rarely exist until the building is completed.

Cast-iron columns are not readily adapted for the design of any other than direct central loading. It is difficult to provide good details, and, in some cases, impossible when the loading is eccentric and there is little rigidity against forces tending to distort the structure. The factor of safety generally used is about 2.7, which is not enough for any columns except those centrally loaded. It would be good practice to limit their use to buildings of 75 ft. in height, and to buildings of greater height only when they have a very broad base.

Section 112 of the New York Building Code prescribes the minimum size of cast-iron columns as 5 ins. and the minimum thickness as $\frac{3}{4}$ in. In the upper stories of buildings of this class, on account of the light loading, it is found cheaper to use light steel sections in preference to cast iron, and single I-beams, channels and two angles, therefore, are often used for columns. Little attention is paid to detailing, excepting to put in connections a sufficient number of rivets for shear and bearing. These columns are little or no better than if cast iron were used, and are sometimes so slender as to make them less desirable. The columns, taken as a whole, from top to bottom are plumb, but zigzag from story to story.

In a building which came under the speaker's observation recently, the equivalent direct load due to an eccentricity of about 16 ins. was more than the direct load on the column.

The following illustration will show to what extent the saving of material is carried. In a building of this character, just now being completed, the girders next to the walls, which were very eccentrically framed to the columns, were, in addition, cantilevered on each side beyond the column, so that the spans of the adjacent girders were reduced by the amount of the projection, and lighter sections were used in these panels.

In another instance, the cast-iron columns were eccentrically loaded simply because the girders had been ordered too short and the column seats and brackets were extended to meet the short girders.

It is, perhaps, a little foreign to the subject, but, in tenement-house construction, even more so than in the apartment hotels, the proper detailing of iron is not more than approximated. In a corner tenement house the entire first story on two sides is left open for stores, and the walls above are supported on girders and columns, the columns resting on piers in the basement. The columns are rarely provided with shoes or caps; the girders simply rest on the columns, and are not bolted to them. Where two sets of girders meet, they are strapped together with a narrow plate placed across the joint

diagonally and bolted somewhere on each side with one bolt, usually the separator-bolt. This joint could open 1 in. without even straining the strap. No attempt is made to tie or anchor the columns, and, when a pier settles, on account of the lack of rigidity, the building partly or wholly collapses. Mr. Whiskeman.

In one case, a narrow column, supporting a front and not provided with a bottom flange or shoe, on account of the concentrated load in the center of a poorly-bedded pier cap, cracked the cap and the front collapsed.

In another instance, practically the whole of a six-story tenement house, in process of completion, collapsed by the undermining of a single pier.

There is practically no engineering at all in tenement-house construction.

The design for the framing of apartment-hotel buildings is a problem for treatment by a competent structural engineer, but is more often solved by the use of tables which are but poorly understood and applied accordingly. In some cases the engineer or architect who knows better will prostitute good design to cheapen the cost.

This is one-half the solution of the problem; the solution of the other half is the employment of a competent superintendent on the premises to see that the work is executed properly and that no undue strains come upon the uncompleted structure.

The solution of this problem is competent engineering on the designs and proper superintendence in the field.

GUY B. WAITE, ASSOC. M. AM. SOC. C. E. (by letter).—The general conditions of construction at the Hotel Darlington appear to be about the same as the average conditions in the construction of most of the apartment hotels erected in New York City at the present time. Mr. Waite.

Since the building laws of New York City have permitted non-bearing walls to be carried on their own foundations, most of these hotels have been constructed with steelwork to carry the floor loads, only.

The steel-floor systems are designed to carry live loads of 60 lbs. per square foot, and dead loads of from 60 to 70 lbs. per square foot, making a total of from 120 to 130 lbs. per square foot.

To carry these floor loads economically, the floor beams are generally spaced at about 6-ft. centers.

The interior columns are studiously located by the architect, who places them as much out of the way as possible, in partitions, etc. The outside columns can usually be located with greater freedom than the interior columns, and may be located to get better results for stiffening the building laterally.

A paper* bearing on the subject of constructing high buildings for

* *Transactions*, Am. Soc. C. E., Vol. XXXIII, p. 190.

Mr. Waite.

resisting lateral forces was presented to this Society in 1894. This paper was presented at a time when the construction of high buildings was comparatively new, and there was no specific building code in New York City regulating them. There were no established precedents among designers for the lateral stiffening of the buildings, and there was no law referring thereto.

The writer, who then had the examination into these constructions for the Department of Buildings, was obliged to act arbitrarily on eccentricities of construction. There were many narrow and high buildings proposed, without, as he believed, safe construction for resisting lateral forces. The discussion which followed this paper applies with equal force to the Hotel Darlington and similar buildings.

In the paper referred to, all parts of the finished structure which were capable of aiding in lateral resistance were considered. Partitions, walls, and steelwork, with the connections, were the principal factors. Partitions are almost insignificant factors in ordinary building construction. The walls and the steelwork are the important factors for lateral resistance, as usually constructed. If the walls are not pierced too much with openings, they, undoubtedly, form the main factors in actual practice.

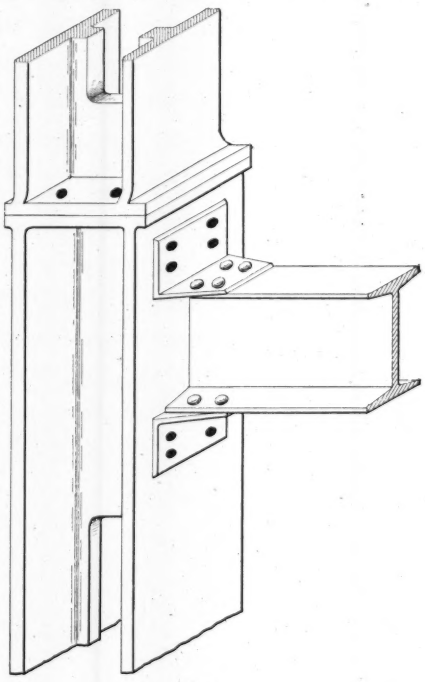


FIG. 7.

Where walls are to supply the principal lateral stiffness to a building, it would seem as though these walls should be carried up with the steel construction, or that suitable temporary lateral braces should be provided until the walls can be carried up to the top of the steel construction.

While walls appear to be generally relied on to furnish the lateral stiffness of buildings, it is believed that the

Mr. Waite.

Hotel Darlington disaster shows the danger of depending too much on this one factor.

The writer believes that the steel construction with its connections should be sufficient to hold the steel skeleton plumb until the walls can be carried up.

As deduced in the former paper, lateral forces may sometimes develop stresses (which increase from the top toward the bottom of the building) which become excessive if the building is very high.

In the Hotel Darlington, as the conditions are understood, the walls not being carried up with the steelwork, the columns and girders with their connections formed the main resistance to lateral forces from the top down to the place where the exterior walls were finished. These lateral forces were probable sufficient at or about the 4th floor (where the wall was stopped) to break the cast-iron columns at their connections.

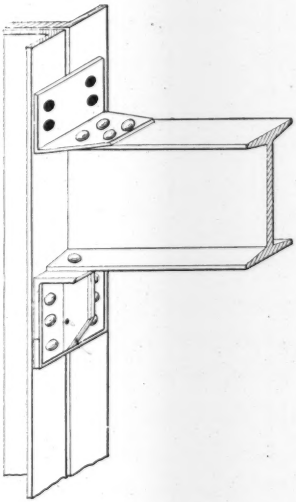


FIG. 8.

The two re-entrant light-court walls, located midway between the front and the rear of the building, were favorable features of construction, adapted to give lateral strength to the narrow way of the building.

Had the structure been held in position until these walls were constructed, and until the floor arches were solid and capable of acting in the capacity of horizontal trusses—conveying lateral stresses to points of greatest resistance—probably nothing extraordinary would have been heard of the Hotel Darlington.

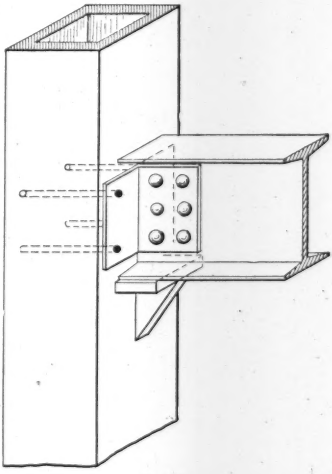


FIG. 9.

Mr. Waite. While the spacing of floor beams and the location of interior columns was bad, in that there was not a direct connection with the columns in every case, this need not be a defect in the construction. Providing other parts of the structure have requisite lateral strength, and the floor system is capable of transmitting the lateral forces to these stiffened parts, these interior columns need only be supporting columns.

The writer now believes, as firmly as he did when preparing the paper ten years ago, that not providing for lateral resistance in steel construction is dangerous.

The writer was associated for several years with iron and steel contractors who were also engaged in the manufacture of cast iron, and, therefore, they desired to use cast-iron columns where practicable. The practice was to use cast iron columns for heavier loads, in lower stories, and steel columns for the lighter loads, in upper stories—the latter being more economical for very light loads than the former.

In order to insure positively rigid connections, for resisting lateral forces, steel connections were riveted to the beams (which were to be connected to columns) in the shop, and were connected to the cast columns, as shown in Figs. 7, 8 and 9, by bolts which could be brought up taut.

The connections shown in Fig. 7 were used in the narrow skeleton-constructed buildings, No. 64 Fulton Street, 25 ft. wide and nine stories high, and No. 708 Broadway, 25 ft. wide and ten stories high, where beams were spaced at about 6.5 ft. centers and were connected at each end to cast-iron I-shaped columns in the lower stories, and to channel-steel columns in the upper stories. Where the floor loads were too heavy for the bolts shown in Fig. 7, connections were made to the column seats as shown in Fig. 8.

Rigid connections have been made to cast-iron columns by shop-riveting angles to the sides of girders, and bolting through the columns, as shown in Fig. 9.

The flange punching for the connections shown in Figs. 7 and 8 is an additional expense, but, where any great resistance is required, the advantages gained will more than compensate for this cost.

Mr. Schneider. C. C. SCHNEIDER, M. AM. SOC. C. E.—This paper illustrates only one of the many examples of poor building construction, and also one of the many abuses of cast iron in such structures.

A careful observer will notice many structures, similar to the Darlington Hotel, which would have collapsed under similar conditions.

In examining the design, the speaker was impressed with the incompetence of the designer, his lack of knowledge of even the first principles of structural designing and his presumption in undertaking a piece of work for which he was not fitted.

The speaker is gratified to know that no member of our profession

nad anything to do with the designing and construction of this struct- Mr. Schneider.
ural monstrosity, but that this design was made by a mechanic whose experience and knowledge of building construction was very limited.

In designing this structure, no attempt was made to insure lateral stability or provision for wind pressure during erection.

The speaker endorses the author's conclusion, that the structure collapsed because of lack of lateral support for the story columns.

The metal framework of a building of the skeleton type of construction is a complete structure in itself during erection, and should be treated as such; that is, it should be self-supporting before the walls and floors are completed, and in that condition be strong enough to resist the wind forces on all exposed surfaces, and such additional strain as may be expected during erection.

The speaker has always adhered to these principles in his own practice, and has found that if provision is made for the usually assumed wind pressure of 30 lbs. on every square foot of exposed surface of the metal framework, the structure is generally strong enough to resist such additional strain as may occur during erection; for instance, the lateral strains produced by a swinging derrick, etc.

There is one feature in the metal structure of the Darlington Hotel which deserves severe criticism, and that is the use of cast-iron columns in a building about 144 ft. high.

Experience has taught us that cast iron is unsafe and unreliable as a material for structural purposes, and, for that reason, it has been entirely discarded in the construction of bridges of all kinds for many years.

In this case, cast iron was used in combination with poor details.

The designer followed the foundryman's practice of carrying the beams or girders on brackets cast on the columns, which, however, the speaker does not consider good practice.

The brackets are generally unreliable, as very frequently they are full of blow-holes, without showing any defect on the outside; this fact is also verified by the author's observation.

In view of these ever-occurring disasters, it seems that something should be done to protect human life against accidents of this kind, which might have been avoided if such structures were properly designed and inspected.

A competent and reputable engineer needs neither building law nor supervision; however, as we cannot compel owners of buildings to employ competent engineers, it becomes necessary to have proper and strict supervision over all structures of this kind.

The people generally rely on the building departments for protection against unscrupulous contractors and ignorant pretenders. Therefore, the building department of every municipality should employ a sufficient number of competent engineers to enable them to

Mr. Schneider. examine the designs properly and carefully and inspect the work of all structures within its jurisdiction.

This supervision should not be perfunctory, but thorough. If a design is not safe, it should be rejected, even if it conforms to the letter of the building law.

It should be the duty of the building department, and that department should have the power to see that all structures are safe, first, last and all the time, with or without building laws.

The indiscriminate use of cast iron should be prohibited. It should be confined to ornamental work, column bases, or such other parts as are only strained in direct compression.

Cast-iron columns should be ruled out in all buildings more than four stories high.

Mr. Lowinson. OSCAR LOWINSON, ASSOC. M. AM. SOC. C. E.—The discussion thus far has demonstrated that certain fundamental errors in construction were committed in the erection of this building. It has also brought out the fact that engineers, apparently, are not agreed as to what constitutes safety in construction.

One sees public departments of a great city, under whose absolute control the construction of buildings is supposed to be, permitting to be erected structures which engineers have stated should not have been built.

The fact that the Darlington was constructed with cast-iron columns has brought out the statement that such a building cannot be built and assurance be given that it will be safe.

Some thirty years ago, because of a disaster caused by the collapse of a bridge, our bridge engineers decided almost unanimously that cast iron has no place among the main structural members of a bridge subject to shock; so that, when we hear prominent engineers state that cast iron, by reason of its unreliability, has no place in a building, there is room for serious reflection and discussion.

The problems connected with apartment hotel construction in New York City are somewhat different from those in commercial buildings. In the first place, the buildings being purely speculative, the designers have been compelled to exercise every device possible to enable them to secure the completion of the buildings at a minimum of cost; and, as the buildings must be sold as soon as possible, there are no responsibilities other than moral, and as long as the building stands together until sold, the only controlling influences are the conscience of the owner and the restriction of the public authorities.

The result of this striving for economy is seen in the forms of construction adopted by structural designers. In buildings of this class, of which there are a great many either lately finished or now in process of construction, cast-iron columns are generally used in the lower stories, and, because of the restrictions of the building code as

to the minimum diameter and thickness of columns, the five or six upper stories of these buildings are usually supported on structural members, which are either I-beams, channels, or, in some cases, even single angles, so that, on seeing such a structure, one cannot comprehend how it is physically possible to transfer the load to these columns, even though they are able to carry it if it were once properly placed upon them. Mr. Lowinson.

A statement of the conditions existing in connection with the construction of buildings of this class, while not pertinent to this question, is deserving of recognition because of its effect upon professional work, and these conditions will be described briefly.

The Darlington is typical, and the owners of the building organized into a company for the express purpose of constructing it. In no way had anyone in this company ever been connected with the responsibility of the construction of a building before.

A firm of architects (whose sole business is to draw plans and make applications to the public departments for the necessary permits for construction) was employed, at a fee which was about one-tenth the schedule rate, adopted by the American Institute of Architects, for this service.

No arrangements whatever were made for superintendence, the owner doing his own superintending.

The owner takes estimate of both labor and materials on each item of the work. In cases where materials will be delivered to a contractor by a firm, the owner tries to get the contractor to supply these (subject to the lowest prices), as he is thereby enabled to shift the responsibility to the contractor. Otherwise, the owner supplies the material and the contractor the labor.

The owner depends on the municipal authorities to see that the labor will be performed properly and usually does not concern himself with them until he receives a notice from one of the public departments that the law is being violated and that he has incurred a financial penalty thereby. His interests are in the finished details, such matters as appeal to the eye and will help make a sale of the building.

It must be borne in mind that the owner's financial interests are usually very little, if any, and he looks to his profits by the increased amount received at a sale over the cost of the structure and land.

In the case in point, it is not worth while to enter into a detailed discussion of any other item than of structural ironwork.

The architects prepared a set of framing plans, and the Bureau of Buildings issued permits, on the assumption that the details did not require examination.

Because of the intimate relationship between the structural ironwork and fire-proof flooring systems, it has become customary for the firms installing the floor systems to prepare a framing plan most

Mr. Lowinson. economical for their system. In this case, the various fire-proof firms easily showed the owners that they could save a sum (in the case of the system adopted) of $6 000 by using different spacing and lighter members in the floor construction.

Of course, there being no engineer or other intelligent person knowing anything about construction, this economy was adopted. (It appears strange that the owners of all these buildings never called their architects to task for this apparent waste of material and money.)

The iron contract for finishing and erecting the building was left to a firm composed of two persons, one of whom knew nothing about iron or buildings, other than would be known by any ordinary layman, and the other had had experience (as he stated at the coroner's inquest) in heating rivets at a forge, and helping screw up bolts on some large buildings where he had been employed previous to going into business for himself. Previous to taking this contract, they had built some fire-escapes and had taken a few small contracts. This was their first real venture.

They realized immediately that it was necessary to have help from somebody who knew something about ironwork, and a man was employed, at a salary of about $15 a week, for this purpose. This man, apparently, had had some experience, and, examining the plans which had been prepared by one of the fire-proof companies, and under which the contract was taken, protested that the floor members were too weak, and he prepared a new set of framing plans in which there was additional iron worth about $2 000. The total value of the contract was in the neighborhood of $28 000.

The cast-iron columns were left as marked on the plans originally. It does not appear that any investigation was undertaken to determine whether some more thousands of dollars might have been saved by scamping these.

The foregoing statement is typical, and there are serious doubts whether there is one building in ten of this class of construction erected under different auspices.

The beams were bolted to girders, girders were bolted to columns, and columns were bolted together, but in no part of the structure was any attempt made to secure the rigidity which is absolutely essential for a structure designed with the formulas used for that purpose; nor was any arrangement made to give assurance that the building must stand under any conditions to which it was likely to be subjected. The connections and the eccentricities of the load were not provided for properly. The flanges of the columns extended on two sides only, so that no safety could be obtained by their resting on secure bases, in case of slight motion, because of possible looseness in connections.

The workmanship in the erection was poor. Mr. Lowinson.

Some three weeks before the collapse, it has been stated by several eye-witnesses, the building was very much out of plumb.

The foreman of the erection gang stated that he believed it was 18 ins. out of plumb.

Apparently, the idea of danger never suggested itself to anyone, even after the leaning was known. To bring it back into plumb, cables and tackles were used. The bolts on the columns of the lower stories were loosened up, and the building was jacked back. There is no record that the columns were screwed up again, and the absence of shims demonstrated that no arrangements had been made for keeping the building plumb, when it had been brought back. Ordinary common sense should have dictated that somebody should be called in to find out the trouble and the remedy, but engineers cost money, and the speculative builder has no money to spend for such foolishness. He has been taking chances all his life, and this is a mere incident. The chances are that luck will not go against him this time.

The side walls of the building were up about three or four stories, the point at which the columns apparently broke off around the walls. The brickwork was laid in cement mortar, and appeared to be fairly well constructed.

The building was what was called a cage construction, which has another grave defect in that the wall columns are embedded in the walls, and, with the projections at their flanges and lugs, permit the wall in shrinking to throw upon the columns and footing considerable weight for which they were not designed.

There is no evidence, however, to show that fracture occurred in any of the wall columns, because of this concentration of the load due to the shrinkage of the walls and the non-shrinkage of the projections on the columns.

The main source of weakness, as stated by several engineers, was the fact that a cast-iron, unbraced building was erected nearly twelve stories in height with practically no provision to resist external strains due to wind and vibration. It had less strength, relatively, than a toy house, twelve blocks high, built with children's playing blocks. Because it had nothing to hold it up, it fell. The same may happen at any moment to any of the other structures of this class.

The speaker believes that the author of the paper states the primal cause of the collapse to be due to the failure of one of the third- or fourth-story columns. It is possible that that may be true. The author evolves a theory of the "Center of Fall," with which the speaker must confess that, because of his unimaginative mind, he cannot bring himself to agree.

Our laws are fairly good. It is a very difficult matter to frame

Mr. Lowinson. laws so as to cover every detail of construction. This building has made manifest a number of weaknesses in the New York Building Code. The great lesson that it is hoped has been learned by this building collapse is the crime of constructing buildings with no intelligent supervision. The idea that a public department should assure the safety of the construction is also wrong. The Bureau of Buildings has certain functions to perform. Its duties are purely those of police surveillance. In Germany, for instance, the Building Inspection Department in each city is a branch of the police and is called "Bau-polizei," Building Police. The officers of this bureau should not be expected to guarantee the absolute safety of the structure, but they should be expected to detect flagrant violations. The Iron Inspector stated that he had about fifty buildings of this class in his district, and spent about ten or fifteen minutes per week at each building.

The main weakness in the law is that there are no means of compelling the class of people who are building now to employ competent men to undertake this construction.

It is an unfortunate fact that many buildings in this city are in danger of collapse at any moment.

Mr. Macdonald. H. P. MACDONALD, JUN. AM. SOC. C. E.—The speaker has been paying especial attention to the ratio of the thickness of metal in cast-iron column flanges and beam seats to that in the shaft of the column, as affecting the solidity of such members. When the metal in such a part is heavier than that of the shaft, it remains in a liquid state after the metal in the shaft is solid, and as it cools and shrinks is likely to draw away, near its center, from the shaft, causing a shrink-hole; such conditions also tend to the formation of blow-holes. The sample which Mr. Parsons has is a good illustration of this case, the metal in the flange before it was machined being probably $1\frac{3}{4}$ ins. thick, while that of the shaft is 1 in. Far more exaggerated cases of this error have come to the speaker's notice, some designers putting beam seats $1\frac{3}{4}$ ins. thick on shafts $\frac{3}{4}$ in. thick. The metal in a flange or bracket should never be more than one-fourth thicker than that of the shaft where it is located; and, where special strength is required, the shaft can be thickened, or two or more vertical ribs be used instead of one, as is the common practice. The same treatment applies to column flanges, which can be made far stronger for a given weight of metal by using vertical reinforcing brackets than by merely thickening the flange. In this case the designer should bear in mind that from $\frac{1}{4}$ to $\frac{3}{8}$ in. must be taken off the flange in machining it, and proportion it accordingly.

The iron in Mr. Parsons' sample appears to be very high in silicon, which would also tend to the formation of shrink-holes. This metalloid should not exceed 2.25% in column castings.

The condition of the column flanges in the Darlington Hotel, as reported by Mr. Parsons, emphasizes the necessity of more technical knowledge on the part of designers and more care in selecting foundries where proper attention is paid to the qualities of the metal used to make structural castings. Mr. Macdonald.

J. H. O'BRIEN, ASSOC. M. AM. SOC. C. E.—The speaker would like to emphasize a point made by Mr. Schneider, in regard to skeleton-constructed buildings, namely, that the skeleton should be designed so that it will carry all loads and resist all external forces likely to come upon it at any time without assistance from the materials which clothe the skeleton. Mr. O'Brien.

The speaker will admit that this principle could not be carried out as effectively with a skeleton, the columns of which are cast iron, as with a skeleton constructed of steel throughout. But, as the previous speakers have disposed of the cast-iron column as unsafe for use in tall buildings, which, usually, are the most important skeleton structures, it may not be amiss at this time to touch upon the need of great care in the design of all important skeleton-constructed buildings.

The speaker has watched the construction of many such buildings, in which the framing is rectangular throughout, and as they grow, tier on tier, without diagonal bracing, or even large gusset plates to stiffen the joints, he has been impressed with their lack of native stability. In most of such cases, of course, the designer expects that external forces will be overcome by the aid of masonry curtain walls and floor systems; but, as these features are seldom erected simultaneously with the steel work, the risk involved, in depending on such assistance, is very great.

In fact, the speaker is aware of one such structure, at least, which was designed and erected by a reputable steel company (which had no control of the mason work), the roof of which collapsed, because it was loaded before the stiffening walls had been built up to reinforce the steel supports.

One of the first principles learned by engineers, who design structures, is that a rectangular frame has, theoretically, no stability, as forces applied along the members of such a frame will change its form. Therefore, the stability which attaches to structures of purely rectangular formation is furnished solely by the stiffness of their riveted connections, which are often designed improperly, usually only for the direct vertical loads.

The columns of many skeleton buildings are so placed, and are designed of such section, that many, and sometimes all, of their loads are eccentric (on the same side of the column); and, too often, no particular attention is paid to this eccentric loading, the effect of which should be offset by adding section to the columns and making sufficient joints at the connections.

Mr. O'Brien. The speaker hopes that Mr. Schneider's point, that skeleton structures should be designed to support all the loads, and to resist all external forces coming upon them, independent of the masonry, will be carried out more faithfully by designing engineers henceforth.

Mr. Just. GEORGE A. JUST, M. AM. SOC. C. E. (by letter).—Faulty design is a sufficient cause for the collapse of the building under discussion. Therefore, it is unnecessary to look for a reason, other than the inherent structural weakness incident to such faulty design.

This being so, it is eminently proper to inquire how the partial erection of such a structure could be possible, in the foremost city of the country under a code especially enacted to safeguard the public interests, as far as they are involved in the construction of buildings.

At the very beginning, a clear distinction should be made between the law, on the one hand, and the administration and interpretation of the law, on the other. The most perfect law, when administered or interpreted by incompetent or corrupt authority, proves ineffectual and often oppressive. But building laws, like all other civil laws even when honestly administered, need intelligent interpretation and consequent modification from time to time, and it is remarkable that, while other branches of law are interpreted and modified by lawyers through the machinery of the courts, public sentiment does not demand that its laws relating to construction be interpreted by engineers, who, it must be admitted, are alone capable of directing the proper application of what is, after all, natural law, as against the law of accumulated precedent.

This inconsistency, however, can be traced to an apparent, if not a real, lack of interest in public affairs by engineers in the past. They, consequently, lack weight when new thought is moulded, even when it relates to matters so distinctly within the engineering field as the formation or amendment of a "building-law."

Now the fact cannot be overlooked that a "building-law" must necessarily be perfectly general in its application; that all its provisions cannot possibly be made so comprehensive as to apply to each specific case, and that any attempt to do so must result in a limitation of individual rights, on the one hand, or endanger public safety, on the other.

And so it may be said that the best criticism made of the New York code, under which the collapsed structure had been partially erected, is that it "pretends too much engineering." The chance of repetition of a "Darlington" disaster would be materially reduced, if much of the present detail matter were eliminated, thus making the provisions of the code more general. The unprincipled practitioner would then not be able, as now, to force a Department of a municipal government to act practically as engineer in the design of work for which he him-

self is incompetent. It might terminate the practice of designing "according to law," and kill the assumption that individual structural members, when of proper section for the performance of their intended functions, will make a safe structure, irrespective of the manner of their assemblage or the quality of the general design. The result should be that experienced professional talent would be engaged more generally. Mr. Just.

As to cast iron: The consensus of opinion, to-day, is undoubtedly favorable to a limitation of its uses, and it should be eliminated from structures of the character of the one under discussion; but even the characteristics and variable quality of cast iron do not call for its utter condemnation. Violent poisons, harmless in the hands of the chemist, prove dangerous playthings for children, and so with cast iron when in the hands of incompetents.

It may be conceded that, in the light of experiment, he permissible unit values, in the New York code, for cast iron in column form, are too liberal. These matters are often the result of compromise, dictated by rival trade interests, strongly intrenched. Engineers, however, are under no compulsion to use such values. It is unfortunate, therefore, that when speculative ventures are involved, or the money for a project is limited, a marked tendency is shown to use the ambiguities and inconsistencies of the law to make the work "cheap."

To this straining for a "cheap" building is also due the present custom of using cast columns in the lower, and steel sections in the upper, stories, with an utter disregard for possible initial bends, eccentric loading or proper transmission. This results in flimsy steel members, which have become an easy target for criticism by those who advocate the continued use of cast-iron columns.

And here it may be noted that the lawyer who avails himself of flaws in the civil and criminal law, to the advantage of his client, enhances his reputation, but the engineer who violates the laws of statics, not only impairs his reputation, but insures the failure of his work.

In recurring to the question of the improper interpretation of the spirit of a law, it can be positively asserted that it was not the intention, of the framers of the New York code, to permit the erection of hybrid structures of the "Darlington" type, and if the decision to the contrary, which has created such a bad precedent, is irrevocable, then the law cannot be too promptly modified in this particular.

It is remarkable how the efforts of the framers were nullified, in this particular. Recognizing the then existing tendency to depart from a pure skeleton type in which the frame should be at all times self-supporting, they intended to recognize only two types, (1) the old, wall-bearing type, and (2) the modern skeleton-frame type of building.

Mr. Just. To distinguish the two types clearly, they demanded for the first time that "wall-girders" be placed continuously at all floor levels in skeleton-frame constructions (Sec. 110). They reduced the then existing minimum thickness of walls for this type (Sec. 36), and removed the heretofore general restriction that front and side walls must be carried up in certain relation to each other, and confined this requirement to wall-bearing buildings (Sec. 41).

No added rigidity, from floor filling or arches, was to be relied upon, for the requirement that the progress of filling in, or floor arching, should bear a certain relation to the height of the frame is not now, and never was, a part of the New York building code. This is a state labor law, enacted presumably for the better protection of workmen engaged on buildings.* The new code also, for the first time, distinctly empowered the head of the Department to call for such structural details as in his judgment might be necessary (Sec. 4).

Therefore, it is unfortunate for good construction that, as was testified by an engineer of the building bureau, before the Coroner's Jury, a joint reading of Sections 36 and 37 was interpreted as a permit to erect buildings of the "Darlington" type. This precipitated upon the public an incomplete and insecure frame, without the restriction that the enclosure walls must be carried up at the same time—removing the only element which might possibly have prevented collapse, as it no doubt has in many cases of equally bad design. The curtain walls named in Sec. 37 were not, in the minds of the framers of the code, intended to include any exterior walls, but referred to non-bearing, interior division walls only.

The present law is weak, in so far as it lacks requirements for proper superintendence, and the "regulations" of the Department on this point are no better. This could be remedied by an amendment requiring that: No building shall hereafter be erected without the continuous direction and superintendence of an architect, or civil engineer, who shall have had at least five years' experience in building construction; no work shall be commenced until the name and address of such superintendent is filed with the Bureau of Buildings, and all changes of superintendent during construction shall be similarly certified to said bureau.

The influence of the American Institute of Architects could be very properly exerted for the enactment of such a section. It would tend to break up the present pernicious practice of architects engaged on speculative work, who accept commissions for the making and filing of plans, leaving the execution of the work to incompetent or unscrupulous persons.

Too often, this early termination of relations between architect and client, is only apparent, and at times results in deceiving the

* Chap. 32, Gen. Laws, Art. 1—Sec. 20, as amended by Chap. 192—laws of 1899.

public authorities. Proof of such continued relationship, however, is shown when these architects, as is often the case, permit themselves to be used to harass the contractors, for the benefit of their speculative employers. Mr. Just.

H. DE B. PARSONS, M. AM. SOC. C. E. (by letter).—The writer was retained by the District Attorney for the purpose of reporting on the cause of the collapse of the Hotel Darlington. Therefore, in the paper, he confined himself to a rehearsal of the facts as found, coupled with his opinion of the cause of the collapse, as contained in his report. Mr. Parsons.

The outside walls along the edge of the lot were continuous from the foundation, with the exception of a portion of the rear wall, which was carried on girders located at the ground-floor level. The walls around the light wells were curtain walls.

The column schedule given in Table 1 does not refer to any particular column. Table 2 gives the sizes of Columns 2, 35 and 36.

The holes in the girders and beams had no regular size, but varied from $\frac{13}{16}$ in. to 1 in. in diameter. It makes very little difference what the exact size of the hole is, as long as it is larger than the bolt, when the effect of lateral stiffness is considered. In every case the bolts were slack in their holes.

The writer has no knowledge of the estimated weights for live load or for dead load which were used by the architect, or by the builder, in calculating the strength of the building.

TABLE 2.—SIZES OF COLUMNS 2, 35 AND 36 OF THE HOTEL DARLINGTON.

Mark.	Column 2.	Column 35.	Column 36.
A	9 x ¾ in.	9 x 1 in.	9 x 1¼ in.
B	9 x ¾ "	9 x 1 "	9 x 1 "
C	6 x ¾ "	9 x ¾ "	9 x 1 "
D	6 x ¾ "	8 x ¾ "	8 x 1 "
E	6 x ¾ "	8 x ¾ "	8 x 1 "
F	6 x ¾ "	7 x ¾ "	7 x 1 "
G	6 x ¾ "	7 x ¾ "	7 x 1 "
H	6 x ¾ "	6 x ¾ "	6 x 1 "
J	6 x ¾ "	6 x ¾ "	6 x ¾ "
K	6 x ¾ "	6 x ¾ "	6 x ¾ "
L	6 x ¾ "	6 x ¾ "	6 x ¾ "
M	6 x ¾ "	6 x ¾ "	6 x ¾ "
N	6 x ¾ "	6 x ¾ "	6 x ¾ "
O	.....	6 x ¾ "	6 x ¾ "

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 979.

THE GATUN DAM.*

By C. D. Ward, M. Am. Soc. C. E.

With Discussion by Charles L. Harrison, M. Am. Soc. C. E.

In a paper,† entitled "Inter-Oceanic Canal Projects," by A. G. Menocal, M. Am. Soc. C. E., it is stated that the Government Commission, appointed in 1875, reported, as to a canal with locks, from Colon to Panama, as follows:

"The river (Chagres) is proposed to be crossed by means of an aqueduct having twelve spans of 90 ft. each, 1 900 ft. extreme length, 65 ft. wide and 26 ft. deep."

In discussing this paper,‡ the late Ashbel Welch, Past-President, Am. Soc. C. E., a thorough and noted canal engineer in his day, said:

"The first thought of an American canal and river engineer, on looking at M. de Lesseps' raised map, is to convert the valley of the lower Chagres into an artificial lake, some 20 miles long, by a dam across the valley at or near the point where the proposed canal strikes it a few miles from Colon, such as was advocated by Mr. C. D. Ward."

The site proposed for this dam was at Gatun, 7.5 miles from deep water at Colon, the end of the canal. But, as is well known, the use

* Presented at the meeting of May 18th, 1904.
† *Transactions*, Am. Soc. C. E., 1879, Vol. VIII, p. 311.
‡ *Transactions*, Am. Soc. C. E., 1880, Vol. IX, p. 148.

of locks was not to be thought of under M. de Lesseps domination, and a sea-level canal was commenced in 1883.

In 1887 it was at last decided that locks would have to be used, and from that time to the present, various locations and elevations for the necessary locks have been suggested, and different sites for the great dam selected.

The New Panama Canal Company and the Isthmian Canal Commission of 1899, the latest authorities, agree in locating the dam near Bohio, 16.5 miles from deep water at Colon, or 9 miles farther up stream than Gatun, the location suggested by Mr. Welch in 1880.

Furthermore, the Isthmian Canal Commission says in its report that "no location suitable for a dam exists on the Chagres River below Bohio."*

The late George S. Morison, Past-President, Am. Soc. C. E., a member of the Isthmian Canal Commission, commenced his paper on the Bohio Dam, which was presented to this Society on March 5th, 1902,† with this sentence:

"All engineers who have examined the route of the Panama Canal agree that the neighborhood of Bohio is the only available location for the dam by which the summit level must be maintained."

Neither of these authorities mentions or gives any consideration to the project of a dam at Gatun, nor even condemns it; nor is the writer aware that any soundings or other examinations, looking to a dam at that point, have ever been made.

As this question may be worthy of more consideration than it seems to have received, some of its advantages may be stated. The canal, as proposed by the latest authority, the Isthmian Canal Commission, may be briefly described as follows: From deep water in Colon Harbor to the Bohio Locks, the Atlantic Maritime Level, cut mostly through the low flat valley of the Chagres, will be 16.81 miles long. The two Bohio Locks, with a lift of 45 ft. each, reach the Bohio Lake, formed by the Bohio Dam, with a maximum elevation of 90 ft. above tide in Colon Harbor and an area of 38.5 sq. miles. The summit level thus attained extends through the Culebra Cut. The distance from the Bohio Locks to the Culebra Cut is 13.61 miles, and from thence through the Culebra Cut to the Pedro Miguel Locks at the farther end of the level the distance is 7.91 miles. The maximum

* "Report of the Isthmian Canal Commission," 1899-1901, p. 90.
† *Transactions*, Am. Soc. C. E., Vol. XLVIII, p. 235.

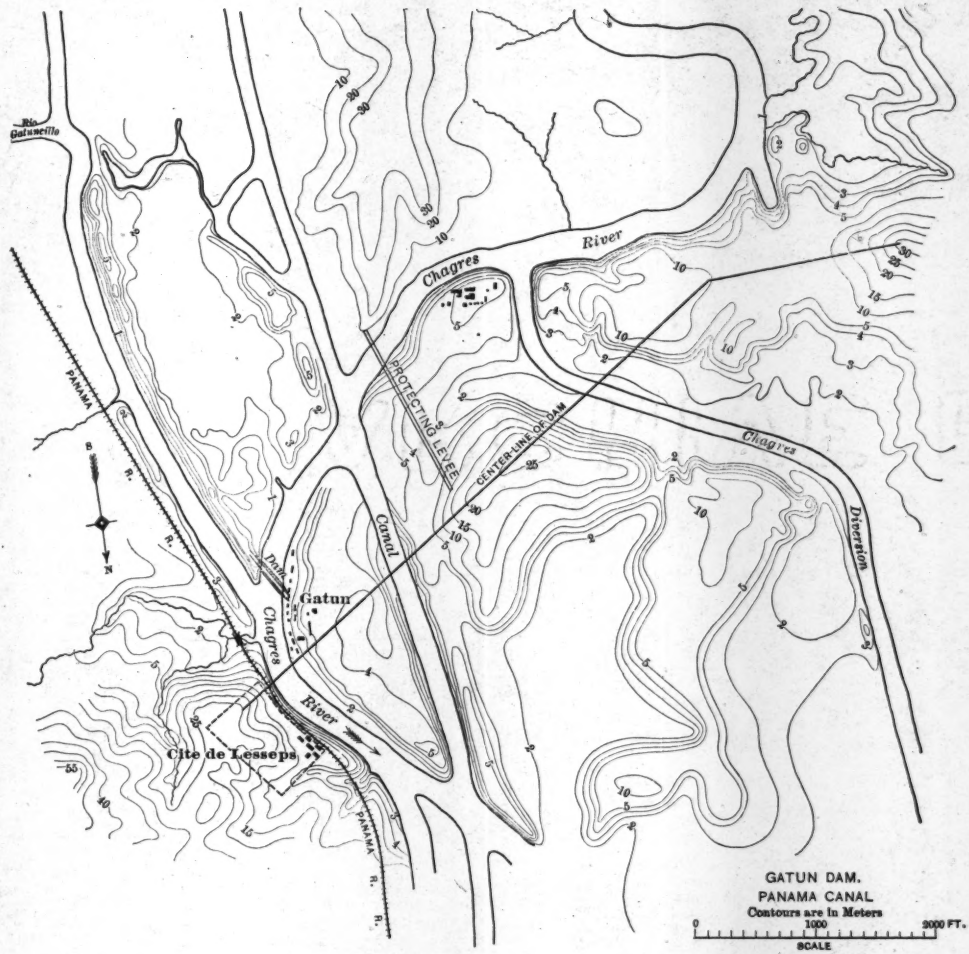
combined fall of these two locks is 62 ft. From Pedro Miguel Locks a level of 1.33 miles extends to Miraflores Lock, which has a maximum fall of 38 ft. to mean low tide in Panama Harbor. For a distance of 4.12 miles beyond this lock the canal extends, through the low swampy country in which the Rio Grande flows, to a point known as La Boca, on the edge of Panama Bay. A dredged channel extends from this point 4.41 miles to the 6-fathom line in Panama Bay.

Allowing 0.90 mile for the various locks, the total length of canal is 49.09 miles.

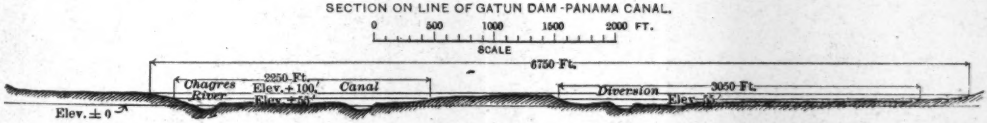
As to the suggested dam at Gatun, the New Panama Canal Company has made a contour map of the region where the dam would probably be located, if at all. This map is reproduced as Plate I, and from it the writer has constructed a section on the proposed center line of the dam. This section indicates the surface of the ground, but, unfortunately, does not show the rock or other solid foundation. If a dam were built here, impounding the water to an elevation of 90 ft., then, of course, the Bohio Dam and the locks and spillway would be omitted, and locks would be built at Gatun.

The length of the dam would be 6 750 ft., while the length of the Bohio Dam is 2 546 ft. The area of the section is nearly three times that at Bohio; therefore, it might not be out of the way, for the present, to say that the cost of the Gatun Dam would possibly be three times that estimated for the Bohio Dam, or $19 108 920. At present, there seems to be no reason to think that the locks would cost more at one place than at the other, but the spillway should be somewhat longer, and might cost more. There may be a question whether the hills to the west of Gatun are high enough to contain the Gatun Lake at an elevation of 90 ft., but the raised map made by the New Panama Canal Company, a reproduction of which is to be seen at Columbia University, indicates sufficiently high ground. The lake would extend far up the valley of the Rio Gatun on the east and the Rio Trinidad on the west, and submerge the Chagres Valley for 9 miles, including the Aqua Clara, Pena Blanca, Vino Tinto and Bruja Swamps, cover 20 or 30 sq. miles of swampy land, and thus improve the healthfulness of the country. This lake area added to the 38.5 sq. miles of the Bohio Lake would be a benefit in every way and would render the Alhajuela Dam on the Upper Chagres unnecessary. Between Gatun and Bohio the excavation of the sea-level canal, the Pena

PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIII, No. 979.
WARD ON THE GATUN DAM.



SECTION ON LINE OF GATUN DAM - PANAMA CANAL.



Blanca Swamp outlet to Aqua Clara Swamp, the Chagres diversion from Aqua Clara Swamp to near Gatun and the diversion of the Rio Gatun, would become unnecessary, and their cost would be saved. These items amount to $14 635 604, a sum large enough to pay the assumed extra cost of the Gatun Dam and leave about $2 000 000 to pay for changing the line of the Panama Railroad and the possible additional cost of the spillway. If these assumptions should prove practicable, the result would be a better and safer canal, shorter time of transit, and more healthy conditions, all at no greater cost and with lower maintenance charges.

If a dam at Gatun, with an elevation of 90 ft., were found inadvisable, one of 45 ft. elevation might be built and the Bohio Dam and Lake retained. Then the lower Bohio Lock would be omitted and established at Gatun and a spillway built there also. The cost of the 45-ft. Gatun Dam might be assumed at $12 000 000, about twice the estimated cost of the Bohio Dam. Its length would be 5 150 ft. The head on the Bohio Dam would be reduced from 90 to 45 ft., and the danger from seepage greatly reduced.

It seems quite possible that with this 45-ft. dam the total cost of the canal would be reduced a few million dollars below the figure named in the report of the Isthmian Canal Commission.

Turning from the Colon end of the Canal to the Panama end, it appears that from the Miraflores Locks to La Boca, a distance of 4.12 miles, the canal is to be excavated at sea level through a low swampy country, with occasional rock, at a cost of $10 963 458. It is suggested, as a plan worth consideration, that this lock be located at La Boca, that a dam and spillway be built there and that the 4.12 miles of swamp be thus flooded, thus affording better and safer navigation than would be had in a narrow channel. The main dam would probably be a mile long, with two or three shorter dams, and might cost no more than the excavation of the 4.12 miles of canal that would be saved. The malarial swamp would be submerged, also. With the 20-ft. rise and fall of tide at Panama, the inrush and outrush of the water in that 4.12 miles of sea-level canal might incommode navigation. All this would be done away with if the last lock was placed at La Boca.

Recurring to the suggested 45-ft. Gatun Dam, with its large lake, the plan of reducing the elevation of the summit level from 90 to 45 ft. may be briefly considered. This elevation is suggested, as 45 ft. seems to be the maximum permissible lift for a lock.

The elevation of 90 ft. for the Bohio Dam seems to have been selected with reference to flood control and in order that the resulting lake should have an area large enough to supply water for lockage, seepage and evaporation during the dry months, without too great a change in the elevation of the surface of the lake.

The 45-ft. dam at Gatun would be so much farther down stream that it might be expected to form as large a lake as the 90-ft. Bohio Dam and perform all the functions of the Bohio Lake.

With one lock at Gatun and one at La Boca, each of 45 ft. lift, the lockage would be reduced to a minimum, and rapidity and safety of transit would be secured. The cost of the modified project might be as follows:

Colon entrance and Harbor	$8 057 707
Colon Harbor to Gatun	942 293
Gatun Dam, Spillway and Locks	19 000 000
Lake Gatun, upper end to Obispo	20 000 000
Obispo gates	295 434
Culebra Section	63 500 000
La Boca Dam, Locks and Spillway	15 000 000
La Boca to deep water	1 464 513
Panama Railroad diversion	1 000 000
	129 259 947
Add 20%	25 851 989
Total	$155 111 936

The estimate made by the Isthmian Canal Commission, with the summit level at 90 ft., was $144 233 358.

The suggestions of 24 years ago have thus been elaborated and presented in this short paper in the hope that, in the discussion, it will appear whether examinations have been made, by any of the various commissions or others, as to the practicability of a dam at Gatun, and, if so, with what result.

If it should appear that such examinations have not been made, it is hoped that this paper will induce those in authority to make such examinations before deciding upon the final plans for the Panama Canal.

DISCUSSION.

CHARLES L. HARRISON, M. AM. SOC. C. E.—Mr. Ward's suggestion that the dam and locks for controlling the waters of the Chagres River and Panama Canal on the Atlantic side be built at Gatun instead of Bohio, is presented for consideration and not as a final project. In the absence of definite information as to the foundations, no attempt is made to give a detailed design, nor to make an accurate estimate of cost. For the purpose of giving an approximation to the cost, the author has made a comparison with the dam at Bohio, as proposed by the Isthmian Canal Commission, which is perhaps the best that can be done in that direction, in the light of present information on the subject. It is very unsatisfactory for the purposes of this discussion to have so little information concerning the character and extent of the material overlying the bed-rock at the proposed location of the dam. As far as the speaker knows, no borings have been made on any part of this line for the purpose of determining the elevation of the bed-rock. However, some comparatively shallow excavations have been made in this vicinity in excavating the Panama Canal and the Diversions for the Gatuncillo and Chagres Rivers. Mr. Harrison.

Starting at the Pacific Coast, near the City of Panama, the general line of the canal runs in a northerly direction to the Atlantic near Colon; but, to avoid confusion in the directions, it will be considered as running north and south, and that part of the Isthmus extending toward South America as being to the east. The range of mountains between the two oceans, and approximately parallel to their coasts, is of volcanic origin. This range of mountains—of moderate height for the entire length of the Isthmus between North and South America—is the only regular feature of the formation. In the vicinity of the Panama Canal the surface of the ground was left in very irregular ridges, valleys and knolls or hills. The Caribbean Sea at one time extended over the entire area of what is now the alluvial swamps of the Chagres River, as far inland as Bohio, where it was connected through a narrow channel with a bay extending over the area approximating in outline the lake which would be formed after constructing the proposed Bohio Dam. To the east of Obispo there existed a large area with no direct outlet, north or south, to either the Atlantic or Pacific Oceans, and the drainage from it flowed west to Obispo and thence north in the lowest valley to the Atlantic. The heavy tropical rains, falling on soil unprotected by vegetation, evidently carried large quantities of silt toward the sea, and resulted in first filling up the valley between Obispo and Bohio and later forming the swamps between Bohio and the sea. This seems to be the probable geological history of the formation of the present Chagres River and the low

Mr. Harrison. lands adjacent to it. It is possible that there may have been considerable subsidence of the coastal territory, as seems fairly well established some distance westward along the Nicaragua Coast. In making the borings at the site of the Bohio Dam, pieces of timber were encountered at a depth of about 100 ft. below sea level, and, in one hole, a log of at least 1 ft. in diameter was found at this depth. Also, in places, sand to a depth of 100 ft. overlies the bed-rock. These data indicate that at one time the bed of the stream corresponded with the present rock surface, and that the entire deposit overlying it was formed in running water.

Between Bohio and the sea a great many knolls of rock project in the ancient valley. Some have their tops about level with the present swamps, others are many feet below it, and still others exist as rocky islands. With such an irregular formation, and in the absence of borings, the depth to rock between these knolls cannot be guessed with any degree of probability. In fact, some of the knolls which project above the swamp are not shown on the existing contour maps of this territory. This is not surprising when the great difficulties of making surveys in that country are known. The low lands are covered with a tropical growth so dense that it is impossible to see into it more than a few feet, and lines must be cut to every point where an elevation is to be taken. It is possible to run a line within 100 ft. of a hill from 30 to 40 ft. high and not discover its existence. Just south of the west end of the proposed Gatun Dam, Plate I, will be seen an excavation starting from the Chagres River. It is related that when this cut was begun it was intended to connect with a tangent of the cut north of the proposed Gatun Dam, and thus form a channel for diverting the waters of the Chagres; but, after the work had progressed for the distance from the river shown on the map, a rocky hill was discovered—the existence of which had not been known before—and the diversion channel was moved further down stream, as shown. A heavy wooded growth, not only makes surveys difficult and expensive, but conceals the irregularities of the ground surface.

Evidently, the existing contour maps were not intended to be made in that detail and extent necessary to give the information for locating a dam to be built to a height of 100 ft. above sea level in the neighborhood of Gatun. It is possible that careful surveys would develop the necessity of building dams or dikes, other than the one shown on Plate I, to impound water at Elevation 90 in the lake. The existence of a proper location for a spillway to discharge the flood waters of the Charges is a very important item. If located near the east end of the dam, it would be expensive and would discharge the water into the canal below the dam, unless this were obviated by constructing for it a separate diversion channel to the sea. A small channel has already been excavated for diverting the waters of the Rio Gatuncillo and the

Mindi, but it would have to be enlarged very much to take the flood waters of the Chagres. By locating the spillway near the west end of the dam, the floods would discharge through the Chagres into the sea without interfering with the canal, but a good location for it is not shown on the maps. Mr. Harrison.

With existing data as to the contour of the country and the character of the foundations, it seems to be impossible to make an estimate of the cost of the Gatun Dam, or to make a satisfactory design. For the purpose of making the estimate of cost, the author has compared its length with that of the dam at Bohio proposed by the Isthmian Canal Commission. This may not be a proper comparison. The length of the Gatun Dam may be greater than estimated, and the depth to rock foundation is an unknown quantity. In the paper* on "The Bohio Dam," by the late George S. Morison, Past-President, Am. Soc. C. E., is given, in Fig. 2, the location of several sections where borings were made. The cross-sections of the valley, on these lines, are shown in Fig. 3. It will be seen that Sections *G*, *B*, *C* and *D* are all shorter than Section *F*, and that the depth of rock on Section *F* is less than on any of the others. In the last-named location the greatest depth to rock is west of the river, and not under its present bed, which, for reasons that need not be given in detail here, was considered a favorable circumstance. Although the depth, even here, was greater than at any other place where foundations of magnitude have been attempted heretofore by the pneumatic process, the adoption of this location for this purpose seemed advisable. The Isthmian Canal Commission, therefore, selected a location where the bed-rock was highest, though it gave a greater length of dam. Doubtless, it was of the opinion that the cost of construction on this line would be less per linear foot than on any of the other lines, and the hazards of construction much less. In view of the unknown depth to good foundations at Gatun, the cost of that dam per linear foot may differ very markedly from the one at the Bohio location. That the cost would be very great, there is no doubt, but the data are not sufficient to make even an approximate estimate.

It is suggested that in case the 90-ft. dam is thought inadvisable, one to impound the water at Elevation 45 might be built. This is open to the same objection as the 90-ft. dam, though the risk might not be as great.

"This elevation is suggested, as 45 ft. seems to be the maximum permissible lift for a lock." Why this is so is not clear. The lock walls can be built just as strong for a 50-ft. lift as for a 45-ft. lift. The uncertainty, if any, must then exist in the lock gates. These would be of metal, and could as well be designed to withstand the water pressure due to a 50-ft. head as for that due to a 45 ft. head. A bridge

* *Transactions*, Am. Soc. C. E., Vol. XLVIII, p. 235.

Mr. Harrison. of 100-ft. span can be designed to carry a load of 100 lbs. per square foot, and one of 500-ft. span can be designed to carry the same load. One would not be considered less safe than the other under this loading. The same reasoning can be applied to lock gates. Under some conditions, the quantity of water used at each lockage might be a controlling factor in determining the lift. If it is assumed that the gates will be wrecked by vessels: such accidents are as likely to happen to one as to the other. The loss of water from the reservoir, when the lift of the lock is greater than the depth of the channel, will be the same in each case, as it is controlled by the elevation of the upper miter sill. It is believed that the lift of the lock need not limit to 45 ft. the height of the alternate dam. The Board of Engineers on Deep Waterways, in June, 1900, reported on a ship canal from the Great Lakes to the Atlantic tide waters, and recommended a lock of 52-ft. lift on that part of the canal between the St. Lawrence River and Lake Champlain. There seems to be no reason why it could not be built and operated successfully.

It is also possible that the advantages, in navigating a broad, shallow channel, over those of a restricted, well-defined channel, are over-estimated.

In all the studies for a high-level canal at Panama, the dam on the Atlantic slope has presented the greatest difficulties, and has seemed to be recognized as the least safe structure in the entire project. If this view is correct, it would be prudent not to increase these uncertainties by building the dam unnecessarily long.

A study of the contour maps and an inspection of the country along the route of the canal show that the narrowest point in the valley is in the vicinity of the proposed Bohio Dam. If the dam were located above this, it would be in a wide basin; and, if below, it would be in the broad swamps of the Chagres.

It is intended in these remarks only to point out more fully the physical conditions, as far as they are known, at the two sites, Bohio and Gatun. It is conceivable, although extremely improbable, that further surface and sub-surface examinations at the Gatun location may reveal conditions making the comparison more favorable to it than the present information would indicate.

The dam proposed at La Boca, near the Pacific terminus of the canal, seems even less advisable than the Gatun Dam.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 980.

ON SEDIMENTATION.*

By Allen Hazen, M. Am. Soc. C. E.

With Discussion by Messrs. Galen W. Pearsons, Robert Spurr Weston, George W. Fuller and Allen Hazen.

Since Seddon published his paper on "Cleaning Water by Settlement,"† there has been but little published discussion on the theory of this subject, but the practice of building and operating sedimentation basins has advanced materially. For example, it has been found in St. Louis that continuous operation, that is to say, a continuous flow of water into, through and out of the basin, gives quite as good results as the intermittent operation which was studied by Seddon, and the new arrangement allows the effluent to be delivered at a higher level, the economical advantage of which is evident. The use of baffles has also been learned, and it has been shown clearly that a well-baffled basin will do as much work as a much larger basin without baffles. A discussion of the subject from a theoretical standpoint, in view of these developments, may lead to a better understanding of it, to the collection of better data, and to improvements in design.

The processes which take place in sedimentation are extremely complex; to discuss them at once in their entirety seems hopeless. First, conditions much simpler than those which actually exist must

* Presented at the meeting of June 1st, 1904.

† *Journal of the Association of Engineering Societies*, 1889, p. 477.

be assumed, and from these simple assumptions the more complex conditions can be approached.

General Assumptions.

Let it be assumed, first, that whenever a particle of suspended matter hits the bottom it remains where it strikes and is never carried forward on the bottom or picked up again; second, that all the sediment in the water is of the same hydraulic value; that is to say, that every particle settles through water at the same rate as every other particle.

Let t = the time required for a particle of sediment to fall from the surface to the bottom of the water in the basin, the water meanwhile being absolutely still;

a = the time of sedimentation in case the action is intermittent; and, in case of continuous operation, let a be the quotient obtained by dividing the capacity of the basin by the quantity of water entering or leaving it during each unit of time;

n = the number of basins, in case several basins are used successively;

x = the proportion of sediment remaining at the end of the process, the amount at the beginning being taken as unity.

Proposition 1.—Assume a basin full of water containing sediment, the water being absolutely at rest and so remaining.

Under these conditions, each particle of sediment will settle toward the bottom at its determined velocity. At the end of a certain period all the particles will have been removed from a top layer of water, which layer will be as thick as the distance that a particle will fall in the elapsed interval, while an amount of sediment equal to that originally contained in the cleared layer at the top will have been deposited upon the bottom. The time required for the removal of all the sediment will be the time required for a particle to settle from top to bottom, or t; and the proportion removed in a shorter period, a, will be $\frac{a}{t}$; and the proportion remaining will be one, less this amount. We then have:

$$x = 1 - \frac{a}{t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The values of x for various values of $\frac{a}{t}$ are plotted in Fig. 1 as Line *A*.

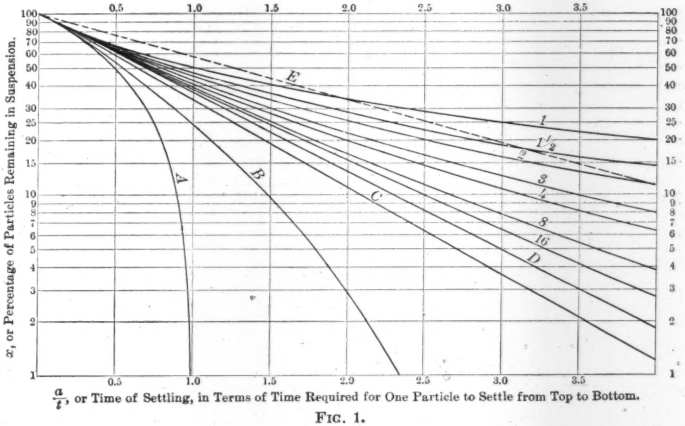


FIG. 1.

In an actual settling basin the water is mixed more or less from top to bottom in the process, with the result that the sediment does not go down in the manner indicated by the assumption. The most important causes of motion are:

1.—The kinetic energy of the water as it enters, which, according to Seddon, is still capable of producing vortex motion after long periods, but which can be much reduced by controlling the water at the entrance so that it has only a very low velocity.

2.—The action of wind (acting in open, but not in covered, basins).

3.—Changes in temperature, which, even though slight, change the specific gravity of the water and produce currents in it.

A development of this proposition, and of the motions of the water, and the resultant mixings, is given in much detail in Seddon's paper.

Proposition 2.—Assume a basin containing water, with sediment as before; and assume that the water is kept mixed during the process of sedimentation to such an extent that the density of the sediment in all parts of the basin is always the same.

In a period of time, $d\,a$, the movement of water in mixing will be inappreciable. The proportion of sediment remaining after the expi-

ration of this period, according to Equation 1, will be $1 - \frac{d\,a}{t}$. Now, the sediment remaining after each subsequent interval, $d\,a$, will be the same proportion of the amount present at the beginning of that interval as it was after the first interval. If, therefore, the proportion remaining after the first interval be multiplied by itself as many times as there are intervals, the amount left at the end of the period will be obtained. The number of intervals will be $\frac{a}{d\,a}$, and we have

$$x = \left(1 - \frac{d\,a}{t}\right)^{\frac{a}{d\,a}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

To solve this, make $a = t$ and develop by the binomial theorem. It is then found that when $\frac{a}{t} = 1$, $x = 0.367878$. The values of x for other values of $\frac{a}{t}$ can be computed from this value by the use of logarithms, and these values are plotted in Fig. 1 as Line *D*.

The mixing assumed is obviously more complete than could exist. To maintain it would require infinite velocities. This condition is impossible, but as it represents one limit of a series, the intermediate terms of which do exist, a consideration of the conditions at the limit aids in the study of actual conditions. If the degree of mixing were less than assumed, the rate of deposition would be more rapid.

Modification of Proposition 2.—Assume that the mixing is such as to keep the density of the sediment in all parts of the lower seven-tenths of the depth of the basin the same, and that in the upper three-tenths of the depth the density of the whole decreases gradually from the full amount to nothing at the surface, the average density being nine-tenths of the density at the bottom.

This assumption agrees reasonably well with the observed densities of sediment at different depths in sedimentation basins. The amount of sediment in the whole basin will then be nine-tenths of what it would be if the density of the sediment at the bottom were maintained to the top, and the density in the bottom layer, from which sedimentation is taking place, will be $\frac{1}{0.9}$ of what it would be if the mixing were complete. The amount of deposit in any interval of time depends upon the bottom density, and would thus be one-ninth faster than it would be with absolutely complete mixing. A given value of x will

then be obtained in nine-tenths of the time that would be required for an equal value of x on the assumption of complete mixing.

This should be corrected slightly, because at the outset the sediment at the top will be as dense as below. The advantage only occurs after the water at the top has commenced to clear. It can be assumed that the time required for this initial cleaning at the top will be 0.1 t, and the gain will amount to 10% of all subsequent time. The equation of this new line is not computed, but is easily laid off graphically, and it is shown on Fig. 1 as Line *C*.

This line may be taken as representing the rate of deposition of sediment on the intermittent system. That is to say, when a basin is filled, allowed to stand until settled, and then drawn off. To compare this line with the results obtained with basins operated with a continuous flow through them, it is necessary to take into account the reserve basin capacity required for the intermittent system of operation. That is to say, the period taken for computation covers only the time that sedimentation is actually taking place. Before this commences, the basin must be filled; and, after it is over, the basin must be emptied. These operations take time. The amount of time out of service will be dependent upon the conditions of service; and, assuming it to be equal to the time of actual use, then twice as much basin capacity will be required, to produce a given result, as is indicated by this line. A line drawn upon this basis on Fig. 1 is marked *E*.

Proposition 3.—Assume a sedimentation basin operated on the continuous plan; that is to say, with water constantly entering at one place and leaving at another. Assume that the water in the basin is kept constantly and perfectly mixed, so that the density of sediment in all parts of it is the same.

As the water in the basin is always perfectly mixed, the density of sediment in the mixture is the same as it is in the effluent, namely, x. In a period, $d\,a$, the proportion of sediment deposited will be $\frac{d\,a}{t}$, and the amount deposited will be $x\left(\frac{d\,a}{t}\right)$. The amount of water entering will be $\frac{d\,a}{a}$, and, as the amount of sediment in this water is unity, the amount of sediment will be $\frac{d\,a}{a}$. The amount of sediment carried by the effluent will be x multiplied by this amount,

or $x\left(\frac{d\,a}{a}\right)$. The amount of sediment deposited must be the difference between that in the water entering and leaving the basin, and we have

$$\frac{x\,d\,a}{t} = \frac{d\,a}{a} - \frac{x\,d\,a}{a}.$$

Solving, we obtain,

$$x = \frac{1}{1+\frac{a}{t}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The values of x computed from this equation are shown in Fig. 1 as Line *1*.

First Modification of Proposition 3.—Assume that the water near the surface carries less sediment than the water in the lower part of the basin, owing to the fact that the mixing is not complete.

Sedimentation will take place more rapidly than is indicated by the equation, and the correction to be made can be computed by the method used under Proposition 2, and, on the same assumptions, it will amount to 10 per cent.

Second Modification of Proposition 3.—Assume that the entering water has a velocity so great as to prevent the deposition of sediment over a certain area.

As far as this is the case, the rate of deposition for the whole basin will be less than computed, in the proportion that the area over which the excessive velocity acts bears to the total area of the basin.

The two corrections suggested by the modifications to Proposition 3 act in opposite ways and tend to balance each other.

Proposition 4.—Assume two basins so arranged that the effluent from one enters the second, all the other conditions remaining as assumed under Proposition 3.

The time period for the first basin becomes $\frac{a}{2}$. The proportion of sediment remaining after passing the first basin will then be, by Equation 3, $\frac{1}{1-\frac{a}{2\,t}}$, and the proportion of this amount remaining after passing the second basin will be the same as it was in the first basin, the amount contained in the water leaving the second basin will then be the square of the proportion in the water leaving the first basin, and we have:

$$x = \left(\frac{1}{1+\frac{a}{2\,t}}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The values of x computed by this equation are shown in Fig. 1 as Line *2*.

Proposition 5.—Assume a series of n basins, the water flowing from each to the next, all other conditions remaining the same.

The computation will then be the same as under Proposition 4, and we have,

$$x = \left(\frac{1}{1 + \frac{a}{n\,t}}\right)^n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

The values of x for three basins are shown in Fig. 1 as Line *3*; for four basins as Line *4*; for eight basins, as Line *8*; and for 16 basins as Line *16*.

If a single basin is very long, and the flow through it is very regular, it is obvious that the ends will have a tendency to act separately; or, in other words, to act as if it were two basins. The formula is general, and we can make $n = 1.5$ for use with such single basins. This line is plotted in Fig. 1 as Line $1\frac{1}{2}$.

Proposition 6.—Assume an infinite number of basins, that is to say, absolutely complete baffling and continuous forward movement of the water at all points, mixing from top to bottom, but with no mixing backward and forward.

The value of n, in Equation 5, becomes infinity, and we have

$$x = \left(\frac{1}{1 + \frac{a}{n\,t}}\right)^n = \left(1 - \frac{a}{n\,t}\right)^n \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

This equation gives the same values of x as Equation 2, and these are represented by Line *D*. In other words, theoretically, a sedimentation basin operated on the continuous system with absolutely complete baffling, would give the same results as a basin on the intermittent system kept absolutely mixed from top to bottom.

Proposition 7.—Assume that there are areas, in corners, etc., where the water remains away from the current passing through the basin, and with little or no exchange with it.

If these areas are entirely out of circulation, they might as well not be there, and sedimentation will take place in the remaining area as if they did not exist. If there is some circulation between these nearly dead areas and the current, they will be of service. If the circulation is sufficient to keep the density of the sediment in the water in these areas up to that in the water in the current at the point where

the current touches the nearly dead area, then sedimentation will be taking place in these areas as actively as elsewhere and no deduction should be made for them. If the circulation is less active than this, the water will contain less sediment than in the current, but still some proportion of it, and the efficiency of the spaces will be in direct proportion to the density of the sediment in the water in them.

For instance, if the whole area of such nearly dead space is 20% of the area of the basin, and if the water in them contains half as much sediment as the water in the current, then 20% of the area is doing half duty; and the whole amount of work done will be that which would be done by a basin nine-tenths as large, and with no such areas.

Proposition 8.—Assume that the water in a basin has, at the bottom, everywhere, or in some places, such a velocity that it will keep the particles moving and hold them in suspension, and will prevent them from being deposited.

If a bottom velocity in a sedimentation basin is such as to prevent the deposition of particles, the work of the basin will be less complete than it would otherwise be. It would seem that deposition would be limited to particles so large as to be capable of being deposited at the existing bottom velocity and that the work of the basin would be limited to the removal of particles larger than this size. This, however, is probably not quite correct, because the velocities in a basin are variable, and even though the average bottom velocities were such as to prevent the deposition of particles of a certain size, there would be areas in the basin where the velocities were below the average, and where these particles might be deposited. The results to be accomplished would thus be represented by the action of a smaller basin, the area of which would be that part of the area of the actual basin in which the bottom velocities were low enough to allow the deposition of particles. The problem is a complicated one, but it seems very clear that bottom velocities should be avoided at all points which would interfere with the deposition of any particles which would otherwise be removed.

If a particle rested upon a smooth bottom and wholly above it, a bottom velocity equal to the velocity at which the particle would settle through still water would exert a pressure on it equal to the weight of the particle, and this would certainly suffice to move it.

On a rough bottom, that is to say, a bottom already covered with particles, many of which might be larger than the particle under consideration, it would seem likely that only a part of the surface of the particle would be exposed, while the resistance to moving would be much greater. Still, it would seem that if the velocity was much in excess of the velocity at which the particle would settle through still water, it would move it. Certainly, if it were moving with the water, it would prevent it from being deposited.

It thus seems a fair conclusion that a bottom velocity approximately equal to the velocity at which a particle would settle through still water would prevent deposition. This statement refers to the velocity within a distance of the bottom measured by the diameter of the particle. That is to say, within 0.1 mm., and even within 0.01 mm. of the bottom and less. Now, nothing whatever is known about the velocities so very close to the bottom. The bottom velocities which can be measured are much farther from the actual bottom. It may be that, with the moderate velocities usually present in sedimentation basins, the action is that of water below the critical velocity, and that the water is attached to the bottom, and has no bottom velocity, and that velocities only commence to exist as the bottom is left, and only become appreciable at distances greater than the diameters of the particles under consideration.

The ordinary mean horizontal velocities in a number of sedimentation basins, to be mentioned later, range from 1 to 8 mm. per second. These velocities are so low that it seems certain that the velocities caused by wind, and perhaps also by temperature changes, will exceed them. As far as this is the case, the controlling velocity would be due to other causes than the general forward movement of the water, and the velocity of this movement becomes of secondary importance.

If the velocities were much greater, the problem would become simpler. It could then be assumed that the bottom velocity was some proportion of the mean velocity, and that the ratio between them was represented by an approximately constant factor, f. Then, the velocity at which particles settle through still water could be taken as, approximately at least, the limiting velocity at which particles would deposit; and, multiplying this by f, would give the mean velocity of the greatest current at which deposition would not be prevented. As-

suming thus a mean velocity, f times as great as the velocity at which the particle settles, the particle would have time to settle from top to bottom of a basin in making a forward movement equal to f times the depth of the basin.

Now, to secure a satisfactory removal of particles with a well-designed sedimentation basin the value of $\frac{a}{t}$ must reach a value which may be taken as approximately 1.5. That is to say, the horizontal course would require to be 1.5 times as long as that course through which water would flow while a particle was settling from top to bottom. This gives the limiting ratio of the longest course which the water is obliged to follow in passing a basin to the depth of the basin at about $1.5\,f$. To make the water follow a longer course than this would prevent the deposition of particles of this size. In other words, to prevent the dragging of particles at the bottom, the length of the course should not be greater than a certain number of times the depth of the basin.

The data at hand for the strength of current which prevents the deposit of sand and gravel particles are not very good. From such data as are at hand it would seem that the value of f is from 20 to 40, and the corresponding length of the course, in proportion to the depth of the basin, is from 30 to 60. This computation, obviously, can be only applied to high velocities and large particles, and it has only a suggestive value in relation to the lower velocities and smaller particles discussed herein.

The conditions which have actually been used are easily learned. There is found in the basins to be mentioned later: At Little Falls, a basin in which the length of the course is but three times its depth; basins at Ithaca and Watertown, with courses about twenty times their depths; basins at St. Louis and Albany, with courses about fifty times their depths; and at Kansas City, a basin with a course more than eighty times its depth. There are other cases, no doubt, particularly where baffles have been used, where the ratios are even higher. It is unfortunate that it is not known whether or not the practice in these cases has been well founded, and whether the basins mentioned present conditions of sufficient repose so that particles stay down when they hit the bottom, or whether the bottom velocities are such as to keep the smallest particles moving even after they hit, and so limit the action to the removal of coarser particles.

Proposition 9.—Assume a very shallow sedimentation basin.

The most common method of expressing the size of a sedimentation basin is to state the length of time that it takes water to pass through it, or, more accurately, the quotient obtained by dividing the capacity of the basin by the quantity of water entering or leaving it in a unit of time. There has been a feeling, which was expressed by Seddon, and by others, that the area rather than the capacity of a basin measures its usefulness. In all the formulas deduced herein, the completeness of removal is a direct function of $\frac{a}{t}$. The effect of the area and depth of basin on the value of this ratio will now be examined.

Let b = the area of a basin;
c = the capacity of a basin;
d = the depth of a basin;
e = the quantity of water treated in a unit of time;
v = the hydraulic value of the sediment, or, in other words, the velocity at which it settles in still water.

Then $a = \frac{c}{e} = \frac{b\,d}{e}$

and $t = \frac{d}{v}$

Combining, $\frac{a}{t} = \frac{\frac{b\,d}{e}}{\frac{d}{v}} = \frac{b\,v}{e}$ .....(7)

In other words, the proportion of sediment removed is a function of the area of the basin and of the hydraulic value of the sediment, and of the quantity of water treated in a unit of time, and is entirely independent of the depth of the basin. This is true of all the propositions which have been considered herein. A very shallow basin will thus do precisely the same work as a deeper one of the same area.

An interesting deduction can be drawn from Equation 7:

$$\frac{a}{t} = \frac{b\,v}{e}$$

Now, $\frac{e}{b}$ represents the upward velocity which would result from the uniform upward flow of the water through the horizontal area of the basin, and the value of $\frac{a}{t}$ can be found by dividing the hydraulic value of the particle, represented by v, by this computed upward

velocity; and conversely, the hydraulic value of particles removed to an extent corresponding to an assumed value of $\frac{a}{l}$ can be obtained by multiplying this computed upward velocity by the assumed value of $\frac{a}{l}$. This gives a means of computing the results which may be expected from a given basin, which does not involve a knowledge of its depth.

The only way in which the depth influences the efficiency of sedimentation is in preventing bottom velocities too great to allow the deposition of sediment. It is obvious that depth has an important bearing in this respect; for, as was shown in Proposition 8 (with some limitations), the longest horizontal length of a course which can be allowed is directly proportional to the depth, and too little depth would limit the size of the basins to impracticably small dimensions.

Proposition 10.—Assume that the water passing through a sedimentation basin flows at the top in a thin layer and that the water below this layer remains quiet or nearly so.

This condition may be caused by a rise in the temperature of the entering water which will tend to keep it at the top. This, practically, has the effect of making a shallow sedimentation basin. But, by Proposition 9, a shallow basin is as effective as a deeper one, as long as the bottom velocities do not prevent the deposition of sediment.

In this case the bottom velocity is upon the still water below, and not upon the actual bottom of the basin. The line between the moving water and the quiet water must necessarily be somewhat indefinite, and the still water below forms an almost ideal receptacle for the particles which settle from the water above. The basin, therefore, will do all the work that it could do if the flow extended to the bottom, and, in addition, the question of bottom velocities is eliminated. This condition seems so attractive as to suggest the desirability of putting low baffles on the bottom of sedimentation basins to hold the water for some distance above the bottom still, or comparatively so, and to confine the current to the upper part of the basin.

Proposition 11.—Assume that the water passing through a sedimentation basin flows in a thin layer upon the bottom and that the water above this layer remains quiet, or nearly so.

This condition may be caused by a fall in the temperature of the

entering water which would tend to keep it at the bottom. This, practically, has the effect of making a shallow sedimentation basin, but the question of bottom velocities is made more difficult, for the current is near the bottom and is more rapid than it would be if the flow were uniform. This condition may thus greatly reduce the efficiency of sedimentation.

Proposition 12.—Assume a sedimentation basin with skimmers to take the water equally from the surface at all points, and an upward flow in the basin to the top, the incoming water being distributed near the bottom.

The first clearing of the water in the process of sedimentation is at the top. It occurs almost instantaneously. If it were possible to remove this water from the top as fast as cleared, and before it had a chance to mix with the more turbid water below, a great improvement could be obtained over the conditions previously assumed.

To make the process effective, it would be necessary to provide skimmers to skim the entire area of the basin. This idea was first expressed in the Rockner-Rothe sewage precipitation tanks, built in Germany, fifteen years ago or more, and an example of which was erected at the World's Fair in Chicago in 1893. A more recent application is found in the Denver Water-Works.* If this arrangement could be developed to its theoretical limit, it would give a removal of sediment corresponding to Line *A*, which is the ultimate theoretical limit. It is not possible to reach this, because even with the best skimming there will be some mixing and consequent drawing from below the surface. The Line *B* is drawn as representing probably as close an approach to the theoretical as could be reached with the best system of skimmers. This line shows clearly the advantage of this process of drawing.

In many cases skimmers have been provided to take water from the top, but such skimmers have usually been located at a point, or at a few points, and, in these cases, unless differences in temperatures prevent, water is drawn to them from all directions, from the bottom as well as from the top, and the skimmer only serves to take the surface water from a small circle in its neighborhood, probably not much greater in radius than the depth of the water.

Proposition 13.—Assume that the sediment, instead of consisting of particles of the same hydraulic value, consists of particles which settle at different velocities.

* *Engineering News*, Vol. 44, p. 322.

If all particles which settle at the same rate be considered as belonging to one class, each class will follow its own law of deposition and will go down at the rate at which they would go if the other particles were not present. In making this assumption all matters of coagulation and flocculation are excluded.

The hydraulic values of particles of different sizes can be taken, and the value of t can be computed for each size. The value of a will be the same for all. The values of $\frac{a}{t}$ can then be obtained, and they will be inversely proportional to the hydraulic values of the particles of various sizes. Taking, then, the curve in Fig. 1 corresponding to the type of basin which is being considered, the corresponding value of x for particles of each hydraulic value can be found; that is to say, the percentages of the particles of those sizes remaining after the treatment. In this way, if a mechanical analysis of the sediment in the raw water were available it would be possible to compute the mechanical analysis of the sediment in the effluent.

Another way is to compute the size of the smallest particles removed by a given set of conditions. To do this it is necessary to assume some percentage of removal which shall be taken as sufficient. This percentage, of course, is arbitrary. It would hardly be taken as less than one-half nor more than seven-eighths. Perhaps three-fourths would answer the purpose. The sizes of particles removed in other proportions will always bear about the same relations to the size selected. That is to say, if the particles of which 75% are removed are taken as having a diameter of 1.00, then 50% of the particles having diameters of 0.78 will be removed, and 87½% of the particles having diameters of 1.47 will be removed. These figures are based on the use of Line *4* and upon Table 1, and are for particles less than 0.03 mm. in diameter. For coarser particles, and with other types of sedimentation basins, the proportion would vary slightly. It will be seen that, considering the great range in size of particles of sediment, the sizes do not vary very greatly, and the size computed in this way may be taken as roughly representing the size of removal. That is to say, it is such a size that particles larger than it will be removed and particles smaller than it will remain. The method of computing this size was stated under Proposition 9.

Proposition 14.—Assume that a sedimentation basin is divided by horizontal plates into two or more compartments, one above another.

As the action of a sedimentation basin is dependent upon its area and not upon its depth, one horizontal subdivision would provide two surfaces to receive sediment instead of one, and would double the amount of work that could be done. Two such divisions would treble it, and so on. If the basin could be cut up by a series of horizontal plates into a large number of shallow passages, the increase in efficiency would be very great. The matter of bottom velocities would have to be carefully looked after, and the ratios of length to depth, the depth in this case relating to each compartment, would have to be maintained. That is to say, the apparatus must be arranged so that the whole process takes place in a short distance of flow, this distance being proportional to one of the spaces between the plates. Such an apparatus may be called a scrubber.

The most serious practical difficulty to be met in carrying out this idea is the method of cleaning. The whole apparatus must be subject to easy and cheap cleaning. Cleaning will be required much more frequently, because, with the depth reduced to a low figure, the quantity of water passed through a given space in a given time would be correspondingly increased, and with it the amount of sediment deposited. If a sedimentation basin 10 ft. deep requires to be cleaned once a year, there would be a corresponding necessity for cleaning a scrubber with spaces of 1 in. between plates 120 times a year, or daily or oftener when the water was turbid.

Proposition 15.—Assume that turbid water is passing through a layer of sand, as in the ordinary process of filtration, and that sedimentation is taking place in all the pores, and that no other influence is at work to remove the sediment.

In this case the sediment is deposited on the surfaces of the sand particles facing upward at places where the current is not strong enough to prevent deposition.

Assume a layer of sand 1 m. thick, with an effective grain size of 0.35 mm., through which water is passing at a rate equal to 3 m. in depth over the surface daily. This rate of filtration is equal to a movement of the water column above the sand of 0.0347 mm. per second. The voids are about 40% of the whole volume. If the pores were all straight and cylindrical from top to bottom, the velocity of the water in them would be $0.0347 \div 0.4 = 0.087$ mm. per second. If 50% is added to represent the increased length of the passages because they are not straight, the velocity would be 0.13 mm. per second.

Experiments have been made in passing solutions of salt and water alternately through sand, which show that in the passage of water through sand a large part of the water in the sand has a low velocity, while a small part has a much higher velocity, so that a part of any given lot of water will make its appearance in a fraction of the time which would be required if the displacement were complete. In other words, some of the passages in the sand are large and comparatively direct, and in them the water moves more rapidly than the average, while the other passages are smaller and less direct, and in these the movement is slower than the average.

If it be assumed that two-thirds of the water in the sand is practically quiet, and that the forward movement takes place in the pores representing one-third of the total volume, the average velocity in these pores will be three times as great as computed, or about 0.4 mm. per second. As the individual pores are, at most, only a fraction of a millimeter in size, it appears that where the velocity is greatest, a given particle of water will pass through a single pore in a period of time which may be roughly stated as a little less than a second. On the other hand, in the smaller pores, where the movement is less rapid, the water may remain in one pore for several seconds.

The velocity of 0.4 mm. per second would not prevent the deposition of large particles of sediment, but such particles are removed at the top by simple straining, and the particles removed by deposition in the sand pores would only be very small particles, and this velocity might be too great to allow this subsidence. In the side passages, however, with lower velocities, conditions for subsidence would be much more favorable. Regarding the sand as a sedimentation basin, it is somewhat analogous to a long series of compartments connected at one side only, with a passageway in which a current is maintained. The bulk of the movement is in the passageway, but there is always some circulation to and from the chambers, which keeps up the supply of sediment particles in them and affords opportunities for sedimentation, and the whole should act as a sedimentation basin, even though the velocity in the channel is too rapid to allow subsidence.

The area of the surface of the sand grains, of sand of the effective size mentioned above, which is about the size commonly used in water filters, is surprisingly large. It is approximately equal to 5 sq. m. per kilogram of dry sand, and the surface area of particles in 1 cu. m. of sand is about 8 000 sq. m. In other words, in a layer of sand 1 m.

thick, the area of the sand particles is 8 000 times as great as the area of the space occupied by the sand.

Assume that, of the total surface area, one-sixth is placed horizontally and facing upward, or near enough to this position to make it capable of receiving sediment. Assume, further, that half of this area is in contact with other sand grains, or too near them to be available. Assume, further, that one-third of the remaining area will be so placed that the water passing it will have a velocity in excess of the rate which would allow the deposition of the finest particles. This reduces the area available for receiving sediment to $\frac{1}{6} \times \frac{1}{2} \times \frac{2}{3} = \frac{1}{18}$ of the whole area of the sand grains, or to an area 444 times as great as the area of the space occupied by the sand. In other words, the opportunities afforded for sedimentation are equivalent to those in a sedimentation basin with an area 444 times that of the filter, or to a basin equal to the filter in area and with 444 horizontal divisions, each capable of receiving sediment. The velocity of a column of water of the same area as the space occupied by the sand amounts to 0.0347 mm. per second. Dividing this by 444, it is found that the upward rate over the area capable of receiving sediment is 0.000077 mm. per second. Assuming (Line *D*) a value of $\frac{a}{t} = 1.4$, with 75% removal, the hydraulic value of the corresponding particles becomes 0.000108, and the diameter of particles corresponding to this (as will be shown in a subsequent paragraph) is 0.0003 mm. In other words, considered as a sedimentation basin, filter sand of the ordinary coarseness and depth, and at about the ordinary rate of filtration, would remove the bulk of all particles more than 0.0003 mm. in diameter. The finest clay is said to consist of particles about 0.0001 to 0.0003 mm. in diameter. A filter would thus fail to remove the finest clay, but would remove all coarser sediment. This conclusion agrees precisely with the facts.

If the rate of filtration is increased, the hydraulic value of the particles which are removed will be increased in the same ratio that the rate is increased; while, if the size of the sand grains is increased, the area of the surface of the particles will vary inversely as the effective size, and the hydraulic value of the particles which can be removed will increase directly with the effective size of the filtering material.

The hydraulic values of the particles which can be removed will thus vary directly with the rate and directly with the effective size of the filtering material. Assuming that the hydraulic value of the particles increases as the square of the diameters, this relation can be stated thus: The diameter of particles removed

$$= c \sqrt{\text{effective size of sand} \times \text{rate of filtration.}}$$

The value of c, in this equation, is found to be approximately 0.0003. The diameters of the particles removed are as follows:

For sand filters of standard construction, sand 0.33 mm. effective size, rate 3 000 000 galls. per acre, the greatest size of particles passing is 0.0003 mm.

If the rate of filtration is increased to 8 000 000 galls. per acre daily, the greatest size of particles passing is 0.0005 mm.

In a mechanical filter without coagulant, and with the same sand, at a rate of 130 000 000 galls. daily, the largest particles passing would be seven times as large as those removed with a sand filter, or 0.0020 mm.

In a scrubber containing gravel, with an effective size of 10 mm., and operated at a rate of 50 000 000 galls. per acre daily, the largest particles passing would be 0.0070 mm.

This size corresponds substantially with the computed size of removal by the sedimentation basins at Albany and St. Louis, and, as far as this computation indicates, a scrubber of this construction would be equivalent in its effect upon the water to the sedimentation basins in use in those cities.

In considering the probable course of sedimentation in sands and gravels one is naturally influenced by knowledge of the action of water in larger spaces and at higher velocities. It must be remembered that in the capillary spaces of the sand the viscosity of the water is of controlling importance, and the principles of flow are different from those with which one is familiar from experience with larger masses.

The purification of water in filters and scrubbers is probably influenced by the flocculation of particles, by surface attractions, by the presence of organisms and, very likely, by other influences of which nothing is known. It is impossible to determine what part, if any, sedimentation plays in the ultimate effect obtained under these conditions, but it is interesting to find that the results computed by the application of the formulas deduced for sedimentation basins give results

apparently in agreement with the facts observed in filtration; and, therefore, it is not irrational to suppose that sedimentation in the pores of these materials may play an important part in the results obtained.

On the Velocity at which Particles of Sediment settle through Still Water.

The larger particles settle rapidly, the smaller ones very slowly. With very small particles the viscosity of water controls, and the velocity of settlement, or the hydraulic value, varies as the square of the diameter. With large particles friction controls, and the velocity or hydraulic value varies as the square root of the diameter. There is a transition space between. This space covers particles from 0.1 to 1.00 mm. in diameter, or ordinary sand, and also extends somewhat beyond these limits.

TABLE 1.—Velocities at Which Particles of Sediment Fall in Still Water.

Diameter of particles, in millimeters.	Hydraulic value, in millimeters per second. 10°C. = 50°F.	Remarks.
1.00	100	Experiments by the writer.
0.80	83	" " " "
0.60	63	" " " "
0.50	53	" " " "
0.40	42	" " " "
0.30	32	" " " "
0.20	21	" " " "
0.15	15	" " " "
0.10	8	" " " "
0.08	6	Interpolated from connecting curve.
0.06	3.8	" " " "
0.05	2.9	" " " "
0.04	2.1	" " " "
0.03	1.3	" " " "
0.02	0.62	Wiley's formula.
0.015	0.35	"
0.010	0.154	"
0.008	0.098	"
0.006	0.055	"
0.005	0.0385	"
0.004	0.0247	"
0.003	0.0138	"
0.002	0.0062	"
0.0015	0.0035	"
0.001	0.00154	"
0.0001	0.0000154	"

NOTE.—These values are not given as being precise, but they are believed to be sufficiently accurate for the purpose of this discussion.

The hydraulic values of particles within these limits have been determined by noting the time required for settlement for a determined

distance through water in a glass vessel. Particles of different sizes were obtained by the methods used in the mechanical analysis of sand. The specific gravity of the particles is about 2.65. The grains are irregular, and the diameters are taken as the diameters of spheres of equal volume.

For particles less than 0.025 mm. in diameter, the formula given by Wiley * is used, namely, $d = 0.0255v^2$, the diameter being in millimeters and the velocity in millimeters per second. The hydraulic values of particles from 0.025 to 0.1 mm. in diameter have been obtained by drawing a curve between the lines representing the higher and lower values. Some of these values are given in Table 1.

On the Effect of Temperature.

The figures in Table 1 are for a temperature of 10° Cent., or 50° Fahr., which is about the annual average temperature of the water in the northern part of the United States. The finer particles settle more rapidly as the water becomes warmer, but with the coarser ones temperature makes less difference. For the finest particles, the rate of settling at different temperatures varies as $\frac{t+10}{60}$, t being the temperature on the Fahrenheit scale. The relative hydraulic values of the same particles at different temperatures are as follows:

Temperature, Fahrenheit.	Relative Hydraulic Value.
32	70
38	80
44	90
50	100
56	110
62	120
68	130
74	140

At a summer temperature of 74°, a particle of sediment will settle twice as fast as at the freezing point. In other words, a given sedimentation basin will do twice as much work in summer as in winter. That is to say, twice as much water can be treated with the same results. Experience indicates the truth of this deduction, and it is also true of filters.

* "Agricultural Analysis," p. 212.

On the Effect of Flocculation and Coagulation.

By flocculation is understood the gathering together of the particles of sediment into aggregates. This takes place more or less with clayey sediment. It is probably not a very important matter in sedimentation basins, because the opportunities for it in rivers before the water enters the sedimentation basins have been so favorable that it has gone as far as it will readily go before the water enters the basin.

Coagulation is artificial flocculation, and is caused by the addition of a chemical to the water. The effect of flocculation or coagulation is to increase the size of the particles, for the aggregates formed in this way have subsiding values which may be very much larger than the subsiding values of their individual particles. Salt water induces flocculation, and the increased rate of sedimentation when fresh water mixes with salt water is to be explained in this way; for it cannot be supposed that an individual particle would settle faster in salt water than in fresh.

Comparison of Different Arrangements of Settling Basins.

The values of $\frac{a}{t}$ necessary to secure the removal of one-half, three-fourths and seven-eighths of the particles of a given size, with basins of different arrangements, and without regard to excessive bottom velocities, are given in Table 2.

TABLE 2.

Description of basins.	Line in Fig. 1.	Values of $\frac{a}{t}$.		
		One-half removed.	Three-fourths removed.	Seven-eighths removed.
Theoretical maximum (cannot be reached)	*A*	0.50	0.75	0.875
Surface skimming, Rockner-Roth system	*B*	0.54	0.98	1.37
Intermittent basins, reckoned on time of service only	*C*	0.63	1.26	1.89
Continuous basin, theoretical limit	*D*	0.69	1.38	2.08
Close approximation to ditto	*16*	0.71	1.45	2.23
Very well baffled basin	*8*	0.73	1.52	2.37
Good baffling	*4*	0.76	1.66	2.75
Two basins tandem	*2*	0.82	2.00	3.70
One long basin, well controlled	*1 1/2*	0.90	2.34	4.50
Intermittent basin, in service half time	*E*	1.26	2.50	3.80
One basin, continuous	*1*	1.00	3.00	7.00

The figures given in Table 2 under the respective headings represent the relative areas of sedimentation basins upon different systems which would be required to produce equivalent results. It will be noticed that the differences between the different systems are always in the same order (with the exception only of the intermittent basin with the allowance for time out of service), but the relative values are different for different percentages of removal. With the removal of one-half of the particles of a given size the differences are less than for the removal of three-fourths of the particles, and for the removal of seven-eighths of the particles the differences are greater.

From the standpoint of preliminary treatment, there would be something to be said in favor of taking a removal of one-half of the sediment of a given size as a standard, because that would represent the bulk of the material; while, from the standpoint of entire clarification, the removal of seven-eighths would be better. The use of the value corresponding to three-fourths removal seems a fair one for a general discussion.

The application of the different lines shown in Fig. 1 to actual sedimentation basins will now be considered. Obviously, the conditions in these basins are more complex than those assumed in the propositions from which the equations of the lines were deduced. Nevertheless, the propositions may represent a sufficiently close approximation to the facts to be of practical use.

Sedimentation basins operated on the intermittent plan seem to meet reasonably well with the assumptions which led to Line *C*, and this line may be taken as representing the conditions in such basins. Sedimentation basins with one compartment and with the water introduced at one point and taken out at another, without any special precautions to distribute it and reduce its velocity, may clearly be taken as being represented approximately by Line *1*. Basins of this type include the St. Louis basins, as they were used two or three years ago, that is, with water entering continuously at one end and flowing out at the other. A sedimentation basin like that at Little Falls, with the water entering at the bottom and taken out at the top at the opposite side, presents a peculiar condition because its depth is so very large in proportion to its length that the removal of the water from the top, even though limited to one side, may be equivalent to a surface straining of a considerable part of its area. Where several

basins are used successively, as at Kansas City and Omaha, the lines having a number corresponding to the number of such basins may be taken directly as representing the conditions. In the case of a very long and narrow basin, the conditions are somewhat different. There is a tendency for the different sections along the length of the basin to act as though they were separate basins. That is to say, there is only a limited amount of circulation between the different parts, and if the process were carried far enough the basin would act as if it were composed of a large number of basins used successively. The sedimentation basins for sewage, designed by Lindley at Frankfort-on-Main, were of this type. Clearly, they are equivalent to several basins, but it is impossible to say how high a number should be assumed. Sedimentation basins for water purification designed upon this principle have seldom been used.

Where the water is introduced very carefully at one end or side of a basin and removed at the other side by a corresponding arrangement, the conditions should be materially improved over those of a single basin, and a line intermediate between *1* and *2* may be used.

Baffling a sedimentation basin consists in building light walls or partitions in it, partially dividing it into a number of compartments. The division is only partial, but it serves in large measure to prevent circulation between the different successive compartments. With very numerous baffles, a sedimentation basin would evidently be cut up so as to be equivalent to a large number of basins used successively. Practically, a basin must not be cut beyond the point which is limited by the allowable bottom velocities, and, except in particular cases, Line *4* may perhaps be taken as representing a practical limit to the results obtained in this way. Under very favorable circumstances, the results indicated by Line *8* might be reached.

Keeping the foregoing classification in mind, the figures in Table 2, for the values of $\frac{a}{t}$ for three-fourths of the sediment removed, indicate the relative areas of basins of the different types required to produce equivalent results. It will be seen that a well-baffled basin, corresponding to Line *4*, will do as much work as a single basin with nearly twice the area, where mixing throughout the whole volume is allowed. Intermittent basins, allowing for the time out of service, do only slightly more efficient work than single basins operated continu-

ously, and, with the most rudimentary baffling, with the continuous system, puts the efficiency above that obtained on the intermittent system.

The theoretical efficiency of the surface skimming is well shown, and only the practical difficulty and expense of carrying out this method limits its use.

Application to a Number of Basins in Use.

The principles and results of these discussions may be better understood by applying them to a number of practical cases. The dimensions have been taken in some cases at approximate figures, and the quantities of water assumed as passing are nominal. In several cases they have been taken as the capacities of the plants, in other cases as the approximate quantities recently treated. The quantity of water treated obviously affects the results materially. In some cases where plants have been operated at much less than their nominal capacities the actual results have probably been correspondingly more favorable.

Several coagulation basins, used in connection with mechanical filters, are included. The primary purpose of these basins is to allow the necessary time for the chemical changes to take place, but the process is facilitated by having them act as sedimentation basins, and it is therefore fair to discuss them in this way. Of course, in these cases, the material removed is of a flocculent nature, very different in its subsiding value from the sand and silt which is taken as the basis of the computation; and the sizes given, therefore, are the sizes of sand particles of equal hydraulic value, and not the actual sizes of the flocculent particles.

Table 3 is a statement of the data, and the computations regarding a number of basins.

The results shown in Table 3 are very striking. The system of reservoirs at Washington should remove particles to 0.003 mm. in diameter, although the horizontal courses are so long that bottom velocities may limit the results. The Kansas City basins remove particles to 0.005 mm. The St. Louis and Albany basins remove particles down to 0.007 mm. in diameter. The results from the coagulating basins at Ithaca and at Watertown are close together, with removals of 0.018 mm. The Warren tank, at Pittsburg, removed particles down to 0.034 mm. and the Jewell down to 0.047 mm. The basin at Little

Falls gives the poorest sedimentation of all, being only able to remove particles more than 0.077 mm. in diameter, which is coarse silt or very fine sand. The latter must be regarded as a coagulation basin, only, and it is obvious that the sedimentation which takes place in it must be very limited.

RÉSUMÉ.

The fundamental proposition, in clearing water by sedimentation, seems to be that every particle of sediment moves downward through the water at a velocity depending upon its size and weight and upon the viscosity of the water. Particles of sediment are generally so far apart that they do not influence each other; and, while there is no doubt that they do sometimes collect in groups and thus change the conditions, it seems to be generally true that each particle will settle as if no other particles were present.

If the water in a basin were absolutely quiet there would be a regular sequence of clearing beginning at the top. The coarsest particles would go down fastest, but at any given point there would be a gradual clearing, and this clearing would take place most rapidly at the top, and, after longer intervals, at lower points in the basin.

Seddon started out with this theory, but found it to be not in accordance with the facts. His observation showed that while the amount of sediment in the water in the top was a little less than in the water in the bottom, the distribution was nearly equal throughout the mass, a condition of affairs inconsistent with the theory. He accounted for this distribution of sediment by the constant mixing of the water from top to bottom, and to the sustaining power of vortex motions in the water. These motions he thought arose from the internal motion of the water at the time of entrance, and from wind, and from temperature changes.

The writer has taken Seddon's development of the case as his starting point, and has carried the discussion further. He believes that while the internal motions keep the water mixed, and with nearly the same density of sediment from top to bottom, the tendency of the particles of sediment to settle is nevertheless an unbalanced force always acting to take the particles to the bottom, and the number of particles that hit the bottom in a given time is proportional, first to the velocity at which the individual particles settle, and second to the density of sediment in the water immediately above the bottom.

TABLE 3.—SEDIMENTATION AND COAGULATION BASINS.

Basin.	Area, in square feet.	Average depth, in feet.	Approximate horizontal course, in feet.	Ratio of horizontal course to depth.	Quantity treated daily, in gallons.	Hours of storage.	Gallons per square foot, daily.	Horizontal velocity, in millimeters per second.	Millimeters in depth of water removed per second.	Type of basin.	Value of $\frac{t}{a}$ for 75% removal.	Hydraulic value of smallest particles removed.	Diameter of smallest particles removed, in millimeters.
East Jersey	5 400	48	130	8	32 000 000	1.30	5 920	8	2.78	Special	2.00	5.56	0.077
Jewell, Pittsburg	131	6.7	.....	.....	250 000	0.63	1 910	.....	0.90	1	3.00	2.70	0.047
Warren, Pittsburg	176	10	.....	.....	300 000	1.05	1 700	.....	0.80	2	2.00	1.60	0.034
Ithaca	4 700	11.4	240	23	3 000 000	3.20	638	6	0.30	4	1.67	0.50	0.018
Watertown	9 880	13.6	270	20	6 000 000	4.00	610	6	0.286	4	1.67	0.48	0.018
World's Fair, Chicago, 1893, sewage	3 136	40	.....	47	2 400 000	9.00	768	.....	0.320	*B*	1.00	0.35	0.015
Albany	220 000	8.5	400		17 000 000	20.00	77	2	0.0983	*1 1-2*	2.33	0.084	0.007
Experimental sand filters, Pittsburg	670	6.8	45	7	34 000	24	51	0.2	0.0240	1	3.00	0.072	0.007
St. Louis	1 600 000	14	750	54	80 000 000	50	50	1	0.0235	1	3.00	0.070	0.007
Kansas City	300 000	20	1 700	85	15 000 000	72	50	2	0.0235	4	1.67	0.020	0.005
Washington	6 690 000	14	8 000	570	75 000 000	225	11	3	0.0053	4	1.67	0.009	0.003

With these fundamental relations in mind, it is easy to compute and to express by simple formulas the proportions of particles of sediment of a given hydraulic value which will hit the bottom under given conditions and which, therefore, presumably, will be removed.

The fundamental propositions may be very concisely expressed. They are: First, that the results obtained are dependent upon the area of bottom surface exposed to receive sediment, and that they are entirely independent of the depth of basin; and second, that the best results are obtained when the basins are arranged so that the incoming water containing the maximum quantity of sediment is kept from mixing with water which is partially clarified. In other words, the best results are obtained where any given lot of water goes through the basin with the least mixing with the water which entered before it, and with the water which enters after it. This is practically accomplished by dividing the basins into consecutive apartments by baffles or otherwise.

Thus far, the discussion is easy and apparently certain. The next step is a more difficult one. It relates to bottom velocities, and has to do with the question whether these velocities are such as to allow the particles to remain on the bottom when they get there, or whether they will be taken up again and be kept in motion with the body of the water. This is a point upon which further experimental data are needed. The problem of securing such data seems to be difficult. The observations must be made at the bottom of a layer of liquid of considerable thickness, where the conditions of observation are not favorable. The observations, further, must be made on very low velocities and on particles so small as to be practically microscopic.

Whatever view may be taken of the second part of the problem, or whatever researches upon it may show, the arrangements of basins most favorable to taking particles to the bottom should stand.

The computations made in this paper show the reasons for several forms of construction already successfully used, and suggest further possible improvements in design.

DISCUSSION.

Mr. Pearsons. GALEN W. PEARSONS, M. AM. SOC. C. E. (by letter).—It is probably correct to say that the waters of our western rivers have been treated more successfully by sedimentation than by filtration, but neither, alone, has been fully satisfactory. Their constantly changing condition, as to high and low water, and the sources from which this water is derived, make the problem a very difficult one. While, most of the year, they may be treated successfully, there are times when they are very obstinate.

At Memphis the writer observed a railroad tank, filled with water from the Wolf River; this, standing unused for three months, maintained its dark color, and, on being emptied, showed no appreciable sediment.

The Kansas River, sometimes for weeks together, shows a really human aversion to settling, and gives the Missouri a similar character. Above Kansas City, the Missouri is not so obstinate; it has bad spells, but not such long ones; the large amount of sediment precludes filtration without previous sedimentation, and, except in some of the smaller towns, the water is little used.

It is stated* that the Missouri carries a maximum of 1 part by weight of sediment in 673. That amount is often exceeded at Kansas City.

In designing the settling basins for Kansas City, in 1874, the writer had the benefit of the experience of the late Henry Flad and T. J. Whitman, Members, Am. Soc. C. E., of St. Louis, and of the late Birdsill Holly, of Lockport, N. Y., who had made careful experiments on both sedimentation and filtration.

The sedimentation at St. Louis was then alternate; Colonel Flad's experiments showed that in still water, of 1 000 parts of sediment, 820 subsided in 6 hours, 900 parts in 12 hours, 930 parts in 18 hours, 945 parts in 24 hours, 966 parts in 48 hours, 969 parts in 96 hours. This was when the condition of the river was favorable for subsidence. In the writer's experiments he seldom found such rapid results, but considers the proportions of sediment carried down in 1, 2, 3, 4, 8 and 16 units of time, as a close approximation.

Mr. Holly's experiments showed that water could be moved at a rate not exceeding 1 ft. per minute, care being taken to make the flow uniform, with but little loss of efficiency in subsidence; and the need of a uniform water level decided the construction of the basins for continuous flow, although their shape was not favorable for the best results. The writer's experience with these has led him to consider continuous flow preferable to intermittent.

* *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 239.

Mr. Pearsons.

While it may be necessary to consider the particles forming the sediment as of uniform hydraulic value, to enable a beginning of mathematical deduction, such uniformity does not exist; observations of the dust falling on a microscope slide will show some particles one hundred times as large as others, and the same, no doubt, is true of the sediment in rivers.

Experiments made by the writer, before designing the settling Kansas City basins, gave results which may be useful in the consideration of the subject.

Clear, flint glass tubes, 3 ins. in internal diameter, were joined with heavy rubber bands, making one tube 5 ft. high, one of 10 ft. and one of 15 ft. These were placed before a tall window, side by side, with the bottoms of the three tubes level, and filled alternately with water from the Kansas and Missouri Rivers, each time filling the three tubes together. Mr. Holly observed some of the experiments. His first observation was that the water cleared quickly at the top, as has been claimed so generally, but, on covering the tops of the tubes, this was seen to be only an appearance caused by light falling on the surface of the water, and no difference in color could be seen between the top and bottom of any of the tubes at any time during the subsidence of the water, the bottom of the 15-ft. tube showing no more color than the 5-ft. tube beside it, at any stage of the subsidence.

This was unexpected, and careful examination was made to discover the reason. At first filling, the water was so turbid that it only showed color. As it cleared gradually the writer was able to see particles descending near the bottom of the 15-ft. tube; at times, something of the same could be discerned in the 10-ft. tube, but with difficulty, and none at any time in the 5-ft. tube.

These particles, by their uniform shape, explained their origin and action; they were pear-shaped, or rather like little tadpoles swimming head down, the tails tapering to invisibility; plainly some larger particle by its quicker descent had overtaken and joined smaller ones, and, increasing by constant addition, had at last become visible, their motion near the bottom being so rapid that, if it had been uniform, but a few minutes would have been required for the whole descent.

These observations induced the belief that more depth could be used in sedimentation than had been considered advantageous, thus increasing the cross-section by depth instead of area. The fragile character of these aggregated particles of soft mud also showed how necessary stillness was to guard against their disruption, and fully bears out all that has been claimed for the use and necessity of baffle partitions.

In 1886 the source of supply was changed from the Kansas to the Missouri River, and continuous flow provided for in the new basins, though their shape made its use less effective than if they had been

Mr. Pearsons. parallelograms. The water was received in a central basin, holding some 8 hours' supply, and provided with a bottom having a slope of 10% to admit frequent and rapid cleaning. From this the water passed through three other basins, each having a capacity of about 1 day's supply (when the flow was from 12 to 14 millions of gallons per day). The receiving basin gave deposits aggregating as much as 30 ft. in depth per year, the second basin about one-third of this depth, and the last only a few inches. Of this deposit, nearly one-half was so thin and soft that it would flow out without washing, and there was no indication in any of the basins of any critical bottom velocity.

The writer dissents entirely from the idea in Proposition 12, that any instantaneous clearing of the surface exists. Though the water at the surface may naturally be considered as freed more quickly from the heavier sediment, its color will be kept to a considerable extent by the lighter material left behind, and the clearing be found more apparent than real. This relates rather to color than weight of sediment, which is doubtless less near the surface.

In Proposition 13, the idea is given that the particles of different hydraulic value simply follow each its own rate of descent, whereas the writer's experiments show that they coalesce with other particles and increase in size as they descend. Although this increase may be only mechanical, it would seem, even when no coagulants are used, that the action may be analagous to that of the formation of rain drops.

Of filtration, Colonel Flad's experiments showed that upward filtration was effective fourteen times as long as downward, with the same water and rate of flow; that is, it required fourteen times as long to clog the filter. This may be considered properly in connection with the subject, for the reason that the upward movement of the water was not greatly in excess of the downward movement of the sediment, and that the flow was under ideal conditions as to stillness in the water treated. Such filtration, in connection with adequate sedimentation, would be a step in advance of any present treatment. This is speaking of the subject in a mechanical light; other considerations may modify these statements, which are not intended as a criticism of Mr. Hazen's able paper, but as presenting some data which may be of service.

Mr. Weston. ROBERT SPURR WESTON, ASSOC. M. AM. SOC. C. E.—At New Orleans the speaker had some experience with small subsiding basins—basins 50 ft. long and 10 ft. deep—and, while the experience gained is not in any way opposed to the formulas, assumptions and propositions in this excellent paper, at many times results were obtained which did not agree with those obtained by Seddon in St. Louis. These opposite results, however, may be explained, perhaps, by differences in local conditions, such as the relative temperatures of air and water.

Mr. Weston.

Much of the Mississippi River water at New Orleans, when introduced into subsiding basins, stratified quickly, especially during the periods when the temperature of the air was higher than that of the water. The incoming turbid water appeared to flow under the water already in the basin, and, leaving part of the suspended matter behind, appeared to rise vertically to the surface layers before it moved toward the outlet.

The inlets of the basins were at one end, 2 ft. from the bottom, and the outlets—overflow weirs the full width of the basin—were at the other. One might argue from certain premises that in these basins the curve of the suspended matter remaining after different periods would reach from inlet to outlet along a curve approaching a parabola, as illustrated by the diagram, Fig. 2.

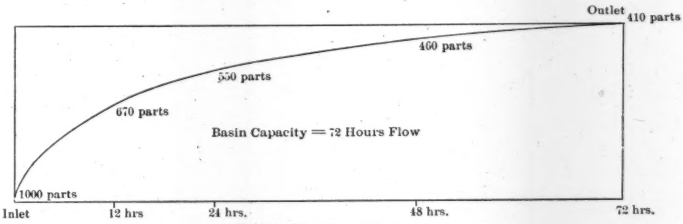


FIG. 2.

At times such conditions were not observed, however, as Table 4* will show.

In Table 4 the increasing temperature of the basin water, during storage, should be noted. This is more of a factor in the South, perhaps, than in other localities, where the temperature of the air would be higher than that of the water for a shorter period during any average year.

At other seasons of the year, at New Orleans, when the temperature of the river water was not as low as that of the air, such differences in turbidity, between the samples taken at different depths, could not be observed.

Baffles are important aids to sedimentation, and, whereas excellent results in subsiding basins are obtained when conditions promote stratification, the speaker believes that the ideal baffle is one which similates stratification, or, as Mr. Hazen has said, a horizontal baffle. In reference to this, it may be interesting to mention the fact that, in water-softening plants, highly increased efficiencies of sedimentation have been obtained by a modification of this design, that is, inclined baffles, inclined so that they may be cleaned.

* "Report on Water and Sewerage," New Orleans, 1903.

Mr. Weston. A trade, water-softening tank, in common use, is an example of this form of construction. In this device the inclined perforated plates are arranged around the wall of the tall subsiding tank; these plates are like shelves one above another. They incline toward the center, which is open. Whenever necessary, the accumulation on the plates is washed to the center of the tank, where it falls to the bottom, whence it may be readily washed out.

TABLE 4.—VARIATIONS OF TURBIDITY AND TEMPERATURE OF WATER IN SUBSIDING BASINS AT VARIOUS DEPTHS.

Date. 1901.	Hour.	Source.	Depth, in feet.	River Water.		Inlet End.		Outlet End.	
				Silica turbidity.	Temperature, in degrees, Fahr.	Silica turbidity.	Temperature, in degrees, Fahr.	Silica turbidity.	Temperature, in degrees, Fahr.
Mar. 26..	10	S. B. 1	0	900	52.1	10	62.6	20	64.4
" " ..	"	" "	2	900	52.1	12	61.7	25	61.7
" " ..	"	" "	4	900	52.1	190	56.3	220	56.3
" " ..	"	" "	6	900	52.1	380	56.3	400	56.3
" " ..	"	" "	8	900	52.1	480	55.4	440	55.4
" 27..	9	" "	0	950	54.6	40	64.4	80	63.5
" " ..	"	" "	2	950	54.6	180	58.1	180	58.1
" " ..	"	" "	4	950	54.6	360	55.4	360	56.3
" " ..	"	" "	6	950	54.6	500	55.4	550	55.0
" " ..	"	" "	8	950	54.6	625	54.6	600	55.0
" 28..	10	" "	0	975	53.2	100	....	100	....
" " ..	"	" "	2	975	53.2	270	....	270	....
" " ..	"	" "	4	975	53.2	400	....	380	....
" " ..	"	" "	6	975	53.2	440	....	480	....
" " ..	"	" "	8	975	53.2	600	....	600	....

NOTE: Silica turbidity results are given in parts per million.

But it is one thing to remove successfully the coarse crystalline sediment resulting from the softening process, and another to remove by such a process the fine colloidal clay particles from natural waters.

It seems to the speaker that the ideal condition for economical subsidence is that outlined under Proposition 12, in the paper, where the water enters near the bottom of the tank and escapes near the top. The nearest approach to this condition would seem to be basins with frequent baffles. These baffles should be built so as to skim the water from the first compartment and deliver it near the bottom of the following compartment. One reason for this is that the frequent bringing of the water to the bottom of the basin, provided always that the velocity is not increased greatly thereby, diminishes the distance through which the particles must sink. It is easier to prevent particles from rising than to precipitate these same particles against the same current.

With this form of construction, it would be easy to arrange to use the whole basin when conditions were such that stratification could

exist, and, if it did not exist, the sedimentation basins would be broken up into compartments, thereby diminishing the effect of wind and temperature disturbances. Mr. Weston.

Mr. Hazen bases a large number of propositions on one value of t. It would be interesting to substitute other values of t,—for instance, infinity. In the latter case, t would be a quantity vastly different from those mentioned in the paper. Mr. Fuller has illustrated this phase of the conception very clearly by noting that part of the colloidal suspended matter, particularly the fine clay in the rivers of the Mississippi Basin, never settles, that is, never from a practical standpoint. This clay is so fine, however, that it is beyond the reach of plain subsidence, and should be treated in other ways. Continuing with this thought along the line of Mr. Hazen's conclusions, one might come to believe that multifold filtration, leaving out of consideration the absorptive power of the sand itself, would not remove continuously and completely the last particles of fine clay, except at prohibitively low rates of filtration.

GEORGE W. FULLER, M. AM. SOC. C. E.—This paper is an excellent one, and the clear-cut manner in which certain portions of this very complex subject are treated should do much to facilitate a clearer understanding of the whole topic. Mr. Fuller.

The first six propositions of this paper, resulting in a series of equations and the data upon which Fig. 1 is based, are treated in a logical manner, on the basis of the assumptions stated. Their treatment from the standpoint of sediment of but a single hydraulic subsiding value is, of course, a matter of convenience, greatly aiding the development of the theoretical side of the subject, to which the paper is essentially devoted. The practical side involves a consideration of ranges in hydraulic subsiding values, t, as they actually occur in the sediment of muddy water, an aspect which the author has taken up in this paper from some, but by no means all, points of view.

The portion of the paper which has interested the speaker most is that considered under Proposition 15. This exposition of the comparative efficiency of filters of different kinds, and of sedimentation basins, in removing suspended matters of various sizes is very ingeniously worked out. The treatment of the ordinary sand filter from the standpoint of a series of minute sedimentation basins should especially clarify the views of those who have held to the German idea, that these filters accomplish their work largely or wholly by biological agencies. It should also prove instructive to those who do not appreciate the limitations of sand filters and scrubbers in removing minute particles of clay and other suspended matter. Mr. Hazen's able presentation of the physics of sedimentation as applied to filtration is in general accord with the results of various studies and investigations in the United States upon water purification, a few of the more general

Mr. Fuller. features of which may be appropriately summarized in this connection as follows:

PHYSICAL ASPECTS OF SAND FILTERS AND SCRUBBERS.

Early Views.—It may be of interest to state that this general physical view of the operation of an ordinary sand filter is by no means new. It was not new when referred to by the late William Ripley Nichols in his lectures more than twenty years ago to the students at the Massachusetts Institute of Technology. He then stated the general features, but not the controlling or comparative details, in a clear manner in these lectures.* Shortly afterward, bacteriology as a science had its origin. At Berlin the subject was studied carefully for several years, largely by medical men, with the result that the biological theory of filtration, taken as a general proposition, occupied in Germany an unduly important position.

"*Schmutzdecke*" *not Indispensable.*—Early German workers were inclined to regard the slimy deposit on the surface of a sand layer as all-important for efficient filtration. Most of them regarded the main body of the sand layer simply as a support and underdrain for the surface film, although some considered the sand layer itself as possessing a steadying influence on the process during abnormal conditions. The early work upon water filtration at Lawrence, Mass., indicated very quickly that the significance of the surface film, while no doubt important, was a factor which had been greatly overestimated. This was shown by the results obtained from numerous test filters treating the Merrimack River water when operating intermittently, which caused the surface film to be disturbed during the daily process of draining and filling the filters. Views similar to those held at Lawrence, as to the overestimated value of the surface coating to sand filters, were advanced by German investigators, particularly Reinsch, of Altona; but, notwithstanding this fact and the large series of observations made at various places throughout this country in dealing with waters of various types, there still seems to exist on the part of many an erroneous impression as to the significance of these surface films. It is true that ordinarily a large percentage of the work is actually accomplished at and near the surface of a sand filter, but the efficiency of filters which have been scraped to an unusual depth, and of filters treating waters containing no constituents to form these surface coatings, presents a line of evidence the soundness of which seems to be beyond all doubt.

Sand Filters with Granular Surface.—In operating filters which receive clay-bearing waters, especially waters of the large rivers of the South and West, from which the sand, silt and coarser clay particles have been removed by deposition in settling reservoirs, one of the

* "Water Supply," by W. R. Nichols, Wiley & Sons, 1883.

most characteristic features is that the sand surfaces, when the water is drawn down preparatory to scraping to relieve clogging, present a granular appearance, and the slimy coating noticed in the filters in Europe and in the northeastern part of the United States is almost entirely lacking. Sand filters receiving such clay-bearing waters possess one important point of similarity to filters operating upon non-clay bearing waters containing organic matter, in that the clay particles detained in and upon the sand layer furnish an adhesive surface which is of importance in causing the detention of the very small particles of clay and of bacteria in the applied water. Mr. Fuller.

Biological Actions not Indispensable.—Many persons seem to regard sand filters as accomplishing their work in the removal of bacteria by the aid of biological agencies, which, in some mysterious manner, cause the destruction of bacteria within the filter. Where filters receive water containing considerable organic matter of an unstable nature, the nitrification process does take place in sand filters in a manner which is very noticeable. On the other hand, sand filters receiving clay-bearing waters quite rich in mineral matter and bacteria, but almost lacking in unstable organic matter, are, under many conditions, also efficient in causing the removal of bacteria. The essential facts in the case are that the bacteria, by physical means, are detained upon and within the sand layer under conditions where they gradually die, or are removed from the surface of the sand layer at times of scraping. While nitrification, in some instances, is of significance in water filtration, it is not essential, as is the case with ordinary sewage filters. Numerous sand filters, which are effective in removing bacteria from clay-bearing waters in the absence of measurable nitrification, afford convincing proof of this point.

Storage and Unloading of Clay Particles.—The physical theory in explanation of the working of sand filters has been much strengthened by experiences in the operation of filters which receive, for many days at a time, water containing a considerable quantity of sub-microscopical clay particles. Under these circumstances, sand filters frequently give a clear effluent and good results in every way for a number of days, but sooner or later their capacity is overtaxed and, for some time, they may produce an effluent which is more turbid, due to clay particles, than the influent. This is, of course, due to the unloading of the stored clay particles from the millions of minute sedimentation basins within the sand layer of the filter. This effect of storage and unloading of minute suspended particles by a sand layer, which was noted at Cincinnati, is also experienced, in some instances, by filters which receive a water containing aluminum hydrate in a fine colloidal form, such as results, in some cases, from the addition of an insufficient quantity of coagulant to allow settling basins, preliminary to filtration, to operate to good advantage.

Mr. Fuller. *Adhesive Surfaces.*—While the *Schmutzdecke* theory has not proved to be generally applicable in explanation of the efficiency of sand filters, it is associated with one idea which is of much importance. Reference is made to the adhesive surface of the films attached to the sand grains, especially near the surface of the layer. These films are formed, of course, in a different way in the case of different waters and sands, and, no doubt, show quite appreciable variations at different times in the case of the same filter dealing with water from the same source in different seasons of the year, etc. It is believed by the speaker that the influence of adhesion resulting from these coatings on the sand grains (or what might be called the floor of the minute sedimentation basins, if one may so speak of the voids within the sand layer), is very important, and, if clearly understood, would go a long way toward explaining differences in the behavior of filters under different circumstances which are at present quite perplexing in some instances. This idea may perhaps be made plainer if this adhesive action should be viewed as offsetting the effect of bottom velocities, such as are apparent in many large settling basins, and which cause the removal with the water of sediment which has once been deposited.

Ripening.—The expression "ripening" of filters, while not perhaps very frequently noted in textbooks, is quite commonly used by those who have to deal with the operation of water filters. When sand layers are new, it is quite generally found, in the case of both sand and mechanical filters, that the efficiencies are somewhat less than normal. General observations indicate quite clearly that this is in many, if not all, cases associated to a considerable degree with the absence of the sticky, gelatinous surfaces which sooner or later become attached to sand layers. Those persons who visited the Lawrence Experiment Station about fifteen years ago will recall that perhaps the most striking feature to be noted there was the purification of sewage accomplished by two test filters of fairly coarse gravel (Nos. 15*A* and 16*A*), which were placed in operation in June, 1889. The voids of these gravel filters contained considerable suspended matter of a more or less sticky nature, and, without doubt, this feature of the filters went a long way in explaining the surprisingly efficient results which they accomplished. In this general connection, mention may be made of the fact that, in a number of instances, mechanical filters have been operated somewhat disadvantageously, due to too thorough cleaning.

Scrubbers.—With regard to scrubbers for a preliminary treatment of certain types of water, preparatory to filtration, the assumption, in this paper, of gravel layers alone, without sponge or other fibrous or fine material, of course lessens their natural efficiency as to the size and relative amount of particles which can be removed, and does not place them strictly in the class with scrubbers such as have been in

operation for some months at the Lower Roxborough filter plant, in Philadelphia, and such as were tested in 1903 at the Ludlow Reservoir, of the Springfield supply. With scrubbers of the type last referred to, there was found to be practically no difference, in the removal of algæ, between them and mechanical filters, operated without the aid of a coagulant, according to the experience gained from several months' investigations in the treatment of the Ludlow water, at Springfield, Mass. Mr. Fuller.

Springfield Tests.—The prime object of investigating these preliminary filters at Springfield was the removal of *Anabæna*, one of the blue-green algæ, which produces seriously objectionable tastes and odors. These organisms are considerably larger than the ordinary bacteria. A feature of much interest and of some surprise was that the percentage of removal of these organisms was less than the removal of bacteria as observed under ordinary circumstances in operating these devices elsewhere. The reason of this is not plain, but very likely is associated with the physical character of the smooth, thin cell walls of these algæ, especially with reference to the comparative absence of a surface which would facilitate adhesion within the pores of a sand layer. Frequently these devices in removing *Anabæna* behaved similarly to sand filters in treating clay-bearing waters in the South and West, as already mentioned, as regards storing and unloading suspended matter.

Practicability of Scrubbers.—The value of scrubbers and other devices for preliminary treatment of water prior to filtration depends upon local conditions, based largely upon elements of cost. In some cases they appear to have much merit; in others, it is doubtful if their efficiency is commensurate with their cost.

Efficiency of Scrubbers.—One feature of rapid-rate preliminary filters which has been clearly demonstrated is their capacity to remove an approximately constant percentage of suspended matter within certain limits, and not to remove clay and other suspended matters, so that there will be a constant and low turbidity in the effluent, regardless of the character of the applied water. The reason of this is that, as the turbidity in river waters increases, following freshets, there is generally a somewhat similar increase of those particles of a hydraulic subsiding value too small to be removed under the given conditions, as set forth in Proposition 15, and as is equally true of plain sedimentation basins.

Comparison of the Practical with the Theoretical Aspects of Sedimentation.

Turning from the phase of the subject dealing with filters and scrubbers to what may be called plain sedimentation, or sedimentation in conjunction with coagulation, the discussion in the paper seems

Mr. Fuller. to be from a rather narrow point of view, in consequence of the premises assumed. While the deductions and statements seem true as general propositions, they are, in some regards, at variance with practical experience as noted by the speaker. This subject is so complex, however, that it is somewhat difficult to set forth clearly the true value to be attached to some observations as regards generalizations. It is believed that the whole subject will take a clearer aspect, if more consideration is given to the wide range in the subsiding value of particles as they are found in muddy waters, and the relation of this point of view to a number of the more important features will be taken up in turn as follows:

Effect of Temperature.—At the bottom of page 64 it is stated that a given sedimentation basin will do twice as much work in summer as in winter, owing to the influence of temperature upon hydraulic subsiding values. This proposition is, in general, true for particles of clay and other matters of such size that their subsiding value is dependent largely upon the viscosity of the water. In practice it is found that, of the wide range in size of particles found in muddy river water a considerable portion have such subsiding values that in almost any settling basin they will become deposited in either summer or winter; there are also in most muddy waters a large number of particles of such size that by plain sedimentation they cannot be removed in any reasonable period of time, even under summer conditions. Therefore the effect of temperature on the operation of sedimentation basins in practice is confined to that range of particles having a maximum subsiding value sufficient to be effective under winter conditions and a minimum value effective only under summer conditions. Within the range of the speaker's observations upon plain sedimentation basins, temperature is not ordinarily a factor of much practical significance. In some cases it is quite noticeable, and, in others, its influence can scarcely be detected.

Dead Spaces in Reservoirs.—Regarding the question of dead spaces in settling reservoirs as an introductory proposition setting forth the advantage of baffles, this matter, as discussed under Proposition 7, may also be amplified from the standpoint of particles having a wide range in subsiding values. It may be of interest here to state that when the Crescent Hill Reservoirs, at Louisville, were cleaned a few years ago, it was found that, except for the coarse material piled up near the inlet, the depth of sediment was substantially uniform over the entire bottom, notwithstanding the fact that the water entered at one corner and passed out at the diagonally opposite corner of these basins, which are nearly square. The only explanation of this, which has occurred to the speaker, is that the relative volumes of water treated per square foot in and near the corners away from the inlet and outlet, as compared with those along and near the most direct line of flow,

were counterbalanced by the relative amounts of sediment deposited per volume of water treated. The fluidity of the deposited sediment may have had some tendency to level the deposits, but it is not believed that this was a controlling factor in these basins, each of which has a capacity of about 50 000 000 gal., or about three days' supply. Mr. Fuller.

Baffles.—Settling basins are provided with baffles or partitions in order to accomplish one of two results. One of these is to produce a velocity of flow of water, so as to prevent deposition of suspended matters and to bring about a thorough mixing of two or more streams, such as is frequently necessary when applying chemical solutions to water or sewage. The other object of baffles is to guard against dead spaces in settling basins, which cause the practical effect of the basin, as an aid to subsidence, to be no greater than that which theoretically would be obtained with a basin of smaller size. Baffles are used, therefore, to accomplish diametrically opposite results, namely, to promote mixing or to prevent deposition on the one hand, and to facilitate sedimentation on the other. No comments are necessary from the standpoint of mixing, as that simply relates to the placing of baffles so as to reduce the cross-section of the moving body of water in order to bring about the desired acceleration in velocity. The use of baffles to facilitate subsidence is undoubtedly a feature which can be used to greater advantage in the United States, under some circumstances, than hitherto has been the practice of American engineers, and, in this regard, the speaker is in accord with the views of the author. Baffles are most advantageously used, as a general proposition, when they make the velocity of the entire flow as nearly equal as possible to that obtained by dividing the contents of the basin by the volume of water passed through it in a given length of time. Baffling to a greater degree than this, except in cases where the mean velocity is much less than the critical velocity, commences to retard deposition by increasing the velocity and making the basin approach a mixing compartment. On the other hand, baffling to a less degree than this theoretically allows dead spaces to exist, thus making the aggregate effect of a basin measurably less than that corresponding theoretically to the dimensions in a given case, as some of the water reaches the outlet too quickly.

Work done by Sedimentation.—In several places in this paper use is made of the expression "work done," as, for instance, it is said that one basin would do twice as much work as another. Those familiar with this subject will have no misunderstanding as to what is meant, but it is quite possible that to the casual reader this may unwittingly be misleading. As basins actually operate in practice, the use of baffles, for instance, to remove dead spaces effectively, does not mean that, as a result of the baffles, twice as much sediment in a given vol-

Mr. Fuller. ume of water will be removed under certain assumed conditions, but it means, in more strict language, that twice as much water may be clarified to the degree in question. In a great many settling basins, the velocity of flow is so low that to remove more silt or clay from the water by plain subsidence is a very difficult and expensive undertaking. There are settling basins where the addition of 100% in size of basin would not allow a removal of 10 or even 5% more sediment from the same volume of water as treated in the unextended basin; and there are also basins which could be modified by baffles, so that 100% more water could be treated with substantially the same degree of clarification.

Economical Limits of Plain Sedimentation.—Extensive investigations at various places in the Mississippi Basin have shown that it is entirely out of the question to remove all the silt and clay from muddy waters by plain sedimentation, and it has become necessary for engineers to recognize that, in water-purification plants, attention must be given to the economical limits of plain sedimentation. Under existing local conditions, an opinion must be formed as to how far it is advantageous and economical to remove sediment by plain subsidence, and how far it is necessary or advantageous to remove it by coagulation, further subsidence and filtration. Obviously, plain sedimentation should not be carried beyond a point where the remaining sediment can be removed more cheaply and just as satisfactorily by other means. This aspect of the proposition shows at once that sedimentation is a much more flexible feature of purification works than many might suppose.

Influence upon Sedimentation of Constructive Features.—As sedimentation basins are actually built, they exhibit about as wide a range of variation in design as is to be found in any ordinary water-works construction, because of the ease with which they may be adapted to important local features and surroundings and, at the same time, serving their purpose within reasonably satisfactory limits. Various sedimentation basins, independent of the very large reservoirs which should be classed as storage basins, range in depth, for instance, from 5 to 50 ft. While there are certain limits which must be provided for as regards their efficiency from the standpoint of sedimentation, it is generally true, at least within the experience of the speaker, that a variety of factors, taken singly or collectively, are more likely to control the most economic and advantageous design than a consideration of the strictly theoretical view-point. Thus, in the projected sedimentation basins at New Orleans, the depth will not exceed about 12 ft., due to the fact that the soil will not safely carry a load of more than 800 to 900 lb. per sq. ft. At Norfolk, Va., the basins adjoining the pumping station have a depth of about 6 or 7 ft., which is as deep as the existing soil conditions conveniently and economically permit in connection with a gravity flow from the storage basins farther up

on the water-shed. The recently built settling basin for the Hacken- Mr Fuller.
sack Water Company, at New Milford, N. J., has a much greater depth and area than will be needed for some time, solely for constructive and economic reasons, due to the fact that, under local conditions, an earth basin, partly in excavation and with earth embankments, could be constructed of much larger size for the same money than was the case with a concrete structure. The result is that for the same money there is obtained a basin far less ideal from the standpoint of the theoretically perfect settling basin, yet one which will adequately serve its purpose (as the capacity of the plant, as a whole, is enlarged) longer than the more ideal concrete structure of equal cost. At Little Falls, the restricted area of the most available site and the desirability of saving the expense of extra pumping caused the construction of a basin of extraordinary depth with reference to its capacity and area. Under conditions still more limited as to area, a deep circular sedimentation tank has been used with satisfaction at Keokuk, Iowa, and tanks of this style are now being built at Burlington, Iowa. In the list of basins given in Table 3 of the paper, there is no doubt about the constructive and economic reasons having been important factors in deciding upon the design of the works. It is needless to recite further instances, as it is obvious that the economical advantages of securing a theoretically perfect settling basin are by no means great enough under many, and perhaps a majority, of conditions to disregard the most careful consideration of a well-balanced design as a whole.

Depth.—There is a good deal to be said of the differences between theory and practice upon the question of depth. Theoretically, depth cuts no figure as long as the vertical transverse section of the mass of moving water is large enough to lower the velocity to that approaching the subsiding value of the smallest particles, which is feasible under the conditions of practice; and, further, provided that the bottom velocities are maintained sufficiently low not to sweep along the bottom and out of the basin too much sediment, especially at times when there is an increase in the volume of water treated by a given basin, with a corresponding acceleration in horizontal velocity. Besides these factors there must also be considered the question of providing for accumulating sediment in a manner consistent with its economical removal. From a theoretical standpoint, there is very little reason for taking serious exception to the statement made by the author that "results are entirely independent of depth"; although, viewing the matter as a practical proposition, it is probably safer to say that depth is the last feature in the design to receive attention for efficient sedimentation. This is the way many European engineers state the matter, and, while they frequently build basins of considerable depth, it is a fact that for sedimentation purposes alone, such as

Mr. Fuller. in chemical precipitation works for sewage treatment, their basins are usually only about 6 or 7 ft. in depth.

Economical Limits in Size of Coagulating Basins.—The first object of a coagulating basin is to afford time for the coagulant to act upon the water, and for the resulting suspended masses to form aggregates of greater subsiding value than the individual particles. The second object is the deposition of the suspended matter. Just as in the case of plain sedimentation, there is an economical limit to the size of coagulating basins. It may be defined by stating that sedimentation in coagulating basins cannot be carried advantageously beyond a point where further removal of suspended matter can be more cheaply or satisfactorily accomplished by filtration. While larger sizes of coagulating basins mark one of the most important improvements of the past decade in water purification, it is also true that this feature can be carried too far, especially in the case of water containing organic matter rather than clay. There are a few filter plants where this latter feature has been carried too far, and the efficiency of the filters materially lessened.

As a practical proposition, the deposition of suspended matter in a coagulating basin need not, and within certain limits ought not, to be carried beyond a point where further removal in the basin means a greater expense for fixed capital charges or enlarged basin construction than the additional cost of attendance and wash water in cleaning filters.

Reference is made in this paper to the coagulating basin of the filter plant of the East Jersey Water Company at Little Falls, N. J. If this basin were operated to remove fine silt and clay by means of plain subsidence it would doubtless be found to be of but little account; but, for the purposes for which it was designed and is operated, that is, to facilitate the coagulation of a non-clay-bearing water and to remove, by sedimentation, a portion of the coagulated masses, it has been found to work very well. As the daily yield of the plant increases, the percentage of wash water will doubtless increase a little, but there is no reason for believing that the cost of extra wash water or of attendance would approximate the fixed charges of enlarging the basin under the very unusual local conditions. Baffles, however, may very likely be worth while later on.

There are two features of the Little Falls plant which are worth mentioning here. One is, that the coagalated masses in the water reaching the filters afford a steadying action upon filtration, which is not obtained with a more completely settled water; and, also, in consequence of this feature, the filters, while washed more frequently than in some cases, can be washed more easily and quickly than is the case with nearly all other filters. Further, it is not necessary to waste any filtered water upon putting these filters in service after washing. The

other feature is the assistance in massing the suspended particles together as a result in part of the upward movement of the water. In this regard, the Little Falls basin resembles the Röckner-Rothe type of basin, such as is discussed by Mr. Hazen, which is similar to the coagulating basins now being built at Burlington, Iowa. Mr. Fuller.

ALLEN HAZEN, M. AM. SOC. C. E. (by letter).—The writer is very much gratified at the discussion. A number of important points have been brought out, and the application of the formulas to practical conditions has been discussed in a way which ought to have the effect of improving the design of sedimentation basins. Mr. Hazen.

Mr. Fuller speaks of two uses of baffles. The writer believes that there is a third use, possibly even more important than the two mentioned by Mr. Fuller, namely, that of keeping the entering water containing the maximum amount of sediment from mixing with the water from which most of the sediment has been removed.

The case of the Louisville Reservoir, as cited by Mr. Fuller, will illustrate this use. Mr. Fuller states that, aside from coarse material piled up near the inlet, the depth of sediment was substantially uniform over the entire bottom, notwithstanding the fact that the water entered at one corner and passed out diagonally at the opposite corner of the basins, which were nearly square.

The writer has repeatedly observed the same state of affairs in other reservoirs, and, in his judgment, it indicates a degree of mixing in those reservoirs such that the water in all parts of the reservoir during the process of sedimentation is substantially of the same quality. That is to say, the conditions are those assumed in Proposition 2. It is unnecessary to repeat here the advantage of baffling and of keeping the partially cleared water separate from that which has not been cleared, as the advantageous results of this procedure are shown equally by theory and practice.

The application of the formulas deduced for sedimentation to the clarification of water by filters and scrubbers was an afterthought, but the development is interesting, and seems to agree reasonably well with the results obtained in practice. As Mr. Weston states, this application indicates that multiple filtration, leaving out of consideration the absorptive power of the sand itself, would not remove continuously and completely the last particles of fine clay except at prohibitively low rates of filtration.

Confining ourselves strictly to the sedimentation theory, and leaving out of consideration the removal of particles by absorption or adhesion, the case may be briefly stated thus:

Water containing sediment may give trouble, when applied to sand filters, in either or in both of two ways. First, excessive amounts of sediment stop the pores of the sand too rapidly and result in short periods of operation between cleaning, and, therefore, in frequent

Mr. Hazen. cleanings, and in expensive and unsatisfactory operation. Second, some particles of the suspended matter may be so fine as to be able to pass entirely through the filter and produce a turbid effluent.

As far as the first point is concerned, there is no question as to the ability of sedimentation to reduce greatly the clogging power of a turbid water, and to increase the length of the periods between cleanings, and, consequently, to reduce the cost of operation of filters; and, further, there can be no doubt that scrubbers may be even more effective in this respect than sedimentation.

Upon the second point, the matter stands quite differently. From the standpoint of sedimentation alone, a turbid filter affluent is produced because there are particles in the raw water so fine that they are not capable of being removed by sedimentation in the pores of the filtering material. A preliminary treatment by sedimentation or by scrubbers removes the greater part of the sediment. The part removed is the coarser part, and the part remaining is the finer part. As stated by Mr. Pearsons, the range between the finest and coarsest may be extremely great. Theoretically, there is no reason why a clearer effluent should be produced with all the coarser part removed, as it is only the finer part which is able to go through. A preliminary process might remove 90 or 99% of the weight of the sediment, but fail to remove precisely those particles which are capable of producing turbidity in the final effluent.

The production of a clear effluent, judged from this standpoint, would seem to depend upon whether or not the raw water contained particles of the degree of fineness to pass the sand, and not at all upon the preliminary processes. Water might contain a thousand, or five thousand, parts per million of sediment, and, if no part of this was of the requisite degree of fineness to pass a filter, a perfectly clear effluent would be produced. On the other hand, if the raw water contained only a very little sediment, if that sediment was fine enough to penetrate the sand, a turbid effluent would be produced. Of course the question remains as to how far the adhesion of the sediment particles for the sand grains and the matters upon them may affect this result.

There are some experimental data in accordance with this view of the case. It was found at Pittsburg, for instance, that with experimental filters it made but little difference in the turbidity of the effluent, whether the raw water was applied directly to the filter or whether the bulk of the sediment was first removed by sedimentation. It did make an important difference in the length of the periods between cleanings.

A consideration of the whole subject leads to the conclusion that where it is necessary to remove such small particles other agencies must be employed, and, fortunately, means are at hand which allow the complete removal of these matters.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 981.

LAKE CHEESMAN DAM AND RESERVOIR.*

By Charles L. Harrison, M. Am. Soc. C. E., and
Silas H. Woodard, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Joseph P. Frizell, Burr Bassell, F. B. Maltby, E. Sherman Gould, E. W. Harrison, Frank C. Horn, Charles S. Gowen, Edward Wegmann, J. Waldo Smith, E. Kuichling, R. Shirreffs, George Y. Wisner, Edwin Duryea, Jr., G. S. Williams, Charles L. Harrison and Silas H. Woodard.

Part I.
HISTORY, DESIGN AND CONSTRUCTION.

By Charles L. Harrison, M. Am. Soc. C. E.

The City of Denver, Colorado, with a population of about 150 000, is supplied with water by The Denver Union Water Company, a corporation which was formed in 1894 by the consolidation of the Citizens Water Company and the American Water-Works Company, both of which had previously been furnishing water to the city and to private consumers.

General Conditions.

Denver is situated about 15 miles from the eastern foothills of the Rocky Mountains, in what is known as the semi-arid region of the West. The elevation of low water in the South Platte River, at

* Presented at the meeting of May 4th, 1904.

Fifteenth Street, is 5 194 ft. above mean sea level, and the higher portions of the city are from 200 to 300 ft. above this.

The precipitation is much less than in the Middle and Eastern States, averaging for the past 32 years only 13.87 ins. Outside of the mountain region, this is principally in the form of rain, but during the winter there is considerable snow, which remains on the ground but a short time. Table 1 gives the monthly and yearly precipitation, from 1872 to 1903. The minimum was 8.48 ins., in 1893, and the maximum 21.43 ins., in 1891, or more than two and one-half times the minimum. In May, 1876, the precipitation was 8.57 ins., and in April, 1900, it was 8.24 ins., or about the same as for the entire year of 1893. In April, 1900, it was greater than for the other eleven months of that year.

TABLE 1.—Record of Precipitation at Denver, Colorado, in Inches.

Year.	January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.
1872	0.55	0.22	0.91	2.09	3.74	2.07	2.69	1.75	1.57	0.68	0.69	0.29	17.25
1873	0.13	0.24	0.22	2.43	0.75	2.24	2.00	1.41	0.89	0.73	0.16	0.61	11.81
1874	0.84	0.53	0.49	1.70	2.43	1.21	3.35	0.68	1.34	0.64	0.08	0.17	13.46
1875	0.38	0.60	0.39	2.24	1.94	0.43	4.17	1.97	2.89	0.22	1.28	0.59	17.10
1876	0.21	0.11	1.80	1.22	8.57	1.10	1.16	2.03	0.60	0.12	1.50	1.70	20.12
1877	1.90	0.40	1.40	2.77	2.30	1.93	0.33	1.30	0.38	2.15	0.73	0.77	16.36
1878	0.10	0.48	1.82	0.05	2.90	2.78	1.33	2.25	1.23	0.80	0.67	1.05	15.46
1879	0.40	0.39	1.00	2.62	3.36	0.32	0.64	1.38	0.02	0.19	0.21	0.33	10.86
1880	0.38	0.32	0.21	0.31	1.11	1.22	1.38	1.46	0.89	1.37	0.83	0.10	9.58
1881	0.50	1.22	0.87	0.50	2.21	0.09	2.50	2.33	0.57	0.32	1.68	0.00	12.79
1882	0.57	0.20	0.20	1.47	2.98	4.96	0.66	1.20	0.06	0.75	0.71	0.73	14.49
1883	2.35	0.45	0.21	3.10	4.30	0.85	2.27	0.75	1.08	1.49	0.32	2.32	19.49
1884	0.22	0.86	0.93	3.33	4.61	1.47	0.65	1.71	0.13	0.21	0.19	0.76	15.07
1885	0.41	0.75	0.97	4.94	2.13	0.66	1.33	1.18	1.22	0.73	0.55	1.08	15.95
1886	0.62	0.72	2.36	2.79	0.09	2.26	0.50	1.62	0.98	0.33	1.93	0.87	15.07
1887	0.67	0.30	0.23	2.16	1.13	0.53	2.49	2.68	0.97	0.97	0.22	0.14	12.49
1888	0.11	0.37	1.15	1.71	2.66	0.29	0.41	1.51	0.11	0.77	0.33	0.09	9.51
1889	0.50	0.70	0.40	1.34	3.44	1.88	2.94	0.33	0.28	2.11	0.53	0.30	14.75
1890	0.18	0.46	0.35	2.50	2.01	Tr.	0.79	1.89	0.17	0.64	0.30	0.04	9.33
1891	1.60	0.27	3.10	2.49	4.15	2.93	0.59	2.84	0.73	0.48	0.69	1.56	21.43
1892	0.40	0.75	1.20	1.75	2.14	1.33	1.19	0.58	Tr.	3.92	0.44	1.32	15.02
1893	0.05	0.83	0.23	0.87	3.09	0.13	1.14	0.35	0.05	0.84	0.55	0.35	8.48
1894	0.18	0.90	0.70	3.30	3.00	0.39	2.11	1.86	1.55	0.19	0.22	0.69	15.09
1895	0.32	0.48	1.19	1.19	2.86	2.65	4.28	0.76	0.98	1.13	0.27	0.01	16.12
1896	0.25	0.24	1.43	0.93	1.27	0.89	2.80	0.97	1.81	0.84	0.10	0.31	11.84
1897	0.58	0.82	0.90	1.31	3.15	2.16	2.06	1.44	0.44	1.64	0.24	0.63	15.37
1898	0.20	0.68	0.28	1.20	4.88	0.94	0.67	0.96	0.28	1.05	0.85	0.99	12.98
1899	0.65	0.58	1.10	0.75	0.15	0.47	1.92	1.78	0.20	1.01	Tr.	0.72	9.33
1900	0.13	0.55	0.63	8.24	0.53	1.87	1.30	0.05	0.87	0.33	0.37	0.42	15.39
1901	0.05	0.06	0.88	1.96	1.18	2.09	0.01	1.30	0.22	0.46	Tr.	0.89	9.10
1902	0.17	0.38	0.63	0.60	1.98	1.89	1.24	0.76	3.70	0.80	0.61	0.59	13.35
1903	0.12	0.42	0.87	0.81	0.75	1.62	1.36	1.35	0.56	1.34	0.07	0.23	9.50
Aver. for 32 years.	0.49	0.51	0.91	2.02	2.56	1.42	1.64	1.39	0.84	0.91	0.54	0.64	13.87

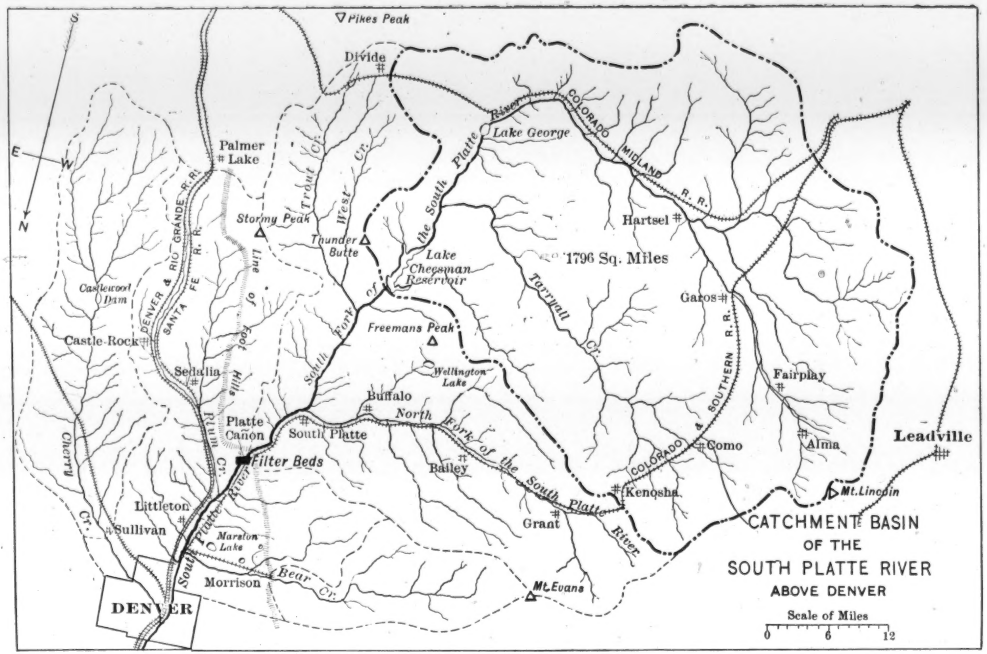

Pikes Peak
Divide
Palmer Lake
Stormy Peak
Thunder Butte
Lake Cheesman Reservoir
1796 Sq. Miles
Lake George
COLORADO MIDLAND R. R.
Hartsel
Garos
Fairplay
Leadville
Como
Alma
Kenosha
Mt. Lincoln
Grant
Bailey
Buffalo
Wellington Lake
Freemans Peak
Castle Rock
Castlewood Dam
Sedalia
Platte Canon
South Platte
Filter Beds
Littleton
Sullivan
Marston Lake
Morrison
DENVER
Mt. Evans
CATCHMENT BASIN
OF THE
SOUTH PLATTE RIVER
ABOVE DENVER
Scale of Miles
0 6 12

FIG. 1

The South Platte River, which flows through Denver, has its source in the Rocky Mountains and, with its tributaries, drains an area above the city of 3 910 sq. miles. About 2 900 sq. miles of this area are in the mountain region and 1 010 sq. miles in the prairies below the foothills. Of the tributaries in the latter area, Bear Creek and Cherry Creek are the only ones which furnish a water supply of any importance.

In the mountain region tributary to the South Platte River, the precipitation is largely in the form of snow, which falls and accumulates from November to April, and melts during May, June and July. The greatest floods in the river occur during these three months, and the magnitude of the floods is in proportion to the amount of accumulated snow in the mountains. During the remainder of the year there are showers and occasional cloud bursts of great violence, but they usually cover small areas and very seldom produce a flood in the river of any considerable extent or duration. Generally, the high-water period is less than two months in duration while the low-water period covers the remainder of the year.

Sources of Supply.

The only source of a sufficient supply of water for Denver is the South Platte River and its tributaries. The ranchmen also draw on these streams for irrigation water. Under existing climatic conditions, the lands will produce very little without irrigation, but with it they become marvelously productive. It is not surprising, therefore, that the ordinary flow of the streams is already over-appropriated by the ranchmen and the water company, so that any increased supply must be secured by storing the flood waters.

The Company secures water from the following sources (see Fig. 1):

1.—Cherry Creek, at Sullivan, where underground cribs are built in the sand and gravel. The water reaches them by infiltration and flows by gravity through a 36-in. pipe to the Capitol Hill Reservoir, from which it is pumped to one of the high-service districts of the city.

2.—In Denver, at Mississippi Street, near the South Platte River, where water is gathered in underground cribs and flows by gravity in a 48-in. wooden pipe to the West Side Reservoir and is then pumped into the distributing mains.

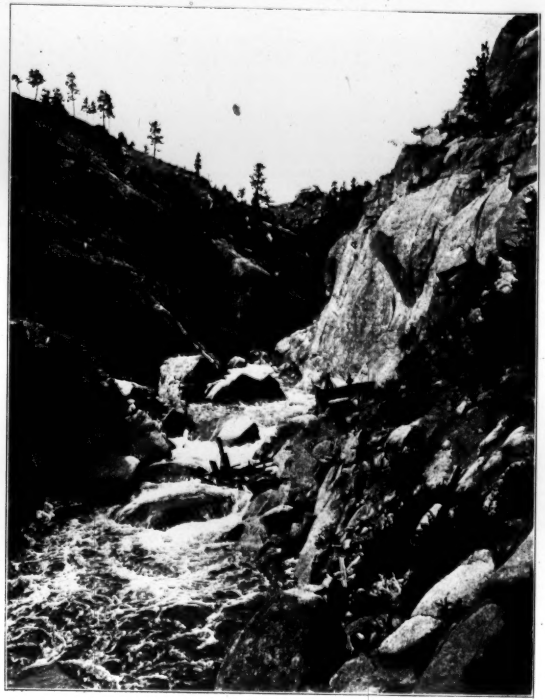


FIG. 1.—VIEW LOOKING UP STREAM AT BEGINNING OF WORK.

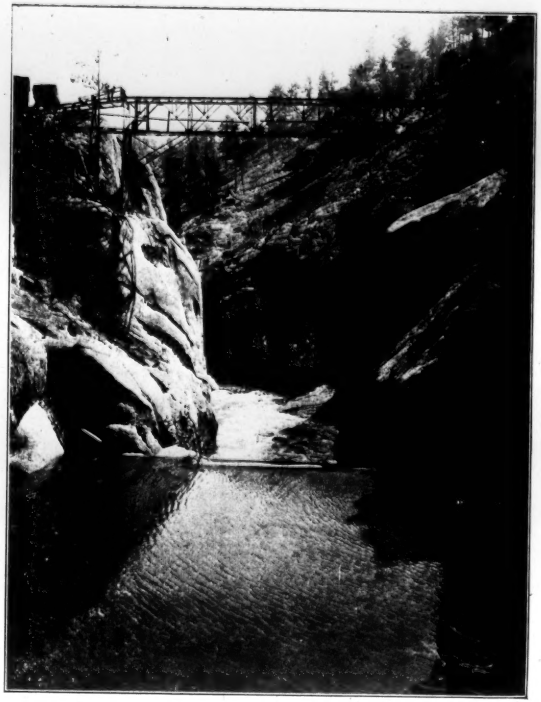


FIG. 2.—SITE OF DAM AFTER FLOOD OF MAY, 1900.

3.—At Morrison, the water is taken from Bear Creek during the flood and non-irrigating seasons and conveyed through the Harriman ditch and a flume to Marston Lake for storage. It is then drawn out as needed for use, filtered through a mechanical filter, and flows by gravity through wooden pipes to the city distributing system.

4.—At Platte Cañon, where the South Platte River emerges from the mountains, a system of underground cribs is constructed to collect the underflow of water in the gravel and sand below the bed of the river and the adjacent valley. The water thus collected flows by gravity through a 30-in. wooden pipe into the Ashland Avenue Distributing Reservoir and into the city pipe system.

5.—About 2 miles above Platte Cañon, water is taken from the river into a 34-in. wooden pipe which leads to a mechanical filter, 2 miles south of Platte Cañon, and, after being filtered, flows by gravity to the Capitol Hill Reservoir and the city pipe system.

6.—In addition to the foregoing, there is now being constructed at Platte Cañon a slow sand-filter plant of about 30 000 000 galls. daily capacity, with provision for extending the filters, as the needs of the city require, to 100 000 000 galls. per day. The water for these filters is secured from the Platte River, under rights owned by the company, and from the Lake Cheesman Reservoir. The supply from the latter source is drawn from the reservoir and turned into the river below and flows in the river bed to Platte Cañon, a distance of about 30 miles, where it is taken from the river into the settling basins and then applied to the filter beds. After being filtered, the water flows by gravity in a 40-in. wooden pipe to the city distributing system.

To convey this water from the various sources of supply to the city requires the use of nearly 100 miles of pipe line, varying from 30 to 48 ins. in diameter. These conditions indicate, in part at least, the great difficulties and expense of securing in this region a water supply for a large city. The great problem is not the best way to distribute the water to the consumers, but how to obtain it.

Lake Cheesman Reservoir.

The first five sources of supply enumerated have furnished sufficient water up to the present time, but the management of the company, having great faith in the future rapid growth of the city,

realized early the importance of securing the necessary water supply for this increased population. With this end in view, Mr. C. P. Allen, the Chief Engineer of the Company, inspected the South Platte River and its tributaries, and, finally, decided upon the location now known as Lake Cheesman as the one furnishing the best site for a dam and large reservoir with a water supply to fill it. The location of this site is in the South Platte Forest Reserve, in Jefferson and Douglas Counties, Colorado, and about 48 miles southwest from Denver (Fig. 1). A subsidiary company, The South Platte Canal and Reservoir Company, was formed for building the reservoir, and, for this purpose, secured the right of way from the Government. The first map was filed on October 8th, 1894, and approved by the Secretary of the Interior on August 19th, 1895. The Company owns about 8 500 acres of land, including and surrounding the reservoir, which ownership insures the protection of this water supply for all time.

The catchment basin, which has an area of about 1 796 sq. miles, is in the Rocky Mountains, on the South Fork of the South Platte River, and is more than 7 000 ft. above sea level. Its location is shown in Fig. 1, a water-shed map compiled from the maps of the United States Geological Survey. In order to make the outlines of the various water-sheds more easily followed by the eye, the map is drawn with the sources of the streams at the top, which brings the north at the bottom instead of at the top of the map, as is the usual custom. The surface of the ground is largely of rock and sand with very little vegetation; and the location in a forest reserve, where settlements which might pollute the waters are excluded, makes the basin an ideal water-shed for gathering potable water. The first habitation on the river above the reservoir is at Lake George, a distance of 25 miles, and the population within the entire area is less than 1.5 persons per square mile. All the streams tributary to the reservoir are precipitous and have rocky beds.

The entire area which will be flooded by the reservoir is of granite or granitic sand, there being no black soil or humus upon it. An inspection of the contour map (Fig. 2) shows that the sides of the reservoir are very steep, thus preventing shallow shore water in which vegetation might grow. The reservoir also has great depth and a comparatively small surface area exposed to evaporation, a considerable advantage in that climate. The source of supply is the melting

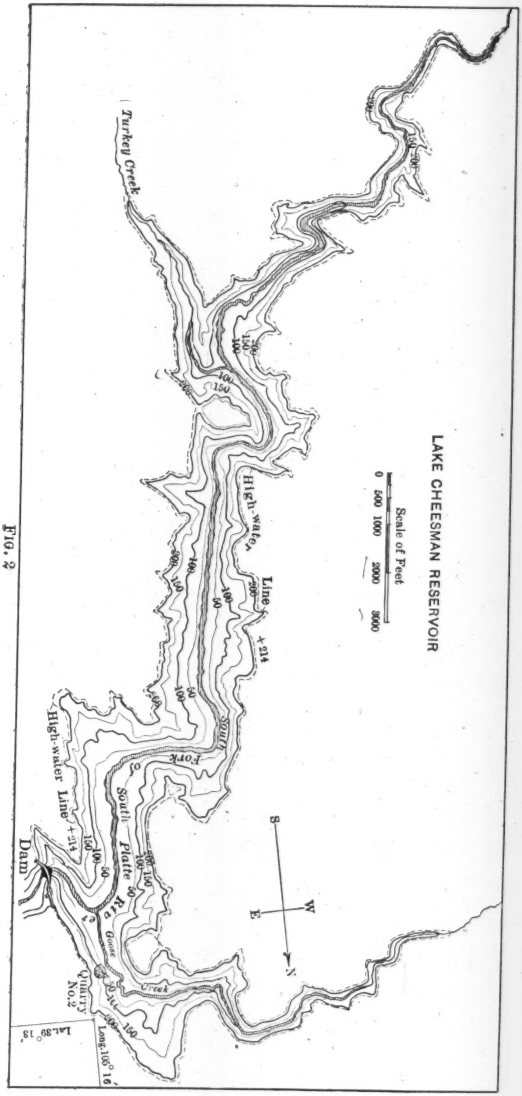

LAKE CHEESMAN RESERVOIR
Scale of Feet
0 500 1000 2000 3000
Turkey Creek
High-water Line +214
South Fork of South Platte River
High-water Line +214
Dam
Goose Creek
Quarry No. 2
Lat. 39° 13'
Long. 105° 16'
S
W
E
N

FIG. 2

snow in the mountains above. Such conditions are unusually good for storing and keeping pure potable waters. The capacity of the reservoir at Elevation 212, which is the crest of the spillway, is about 3 500 000 000 cu. ft., and the surface area is about 874 acres. See Fig. 3.

The Rock-Fill Dam.

After the company secured from the Government the right to build the dam, it was necessary to obtain the ownership of certain claims, taken up by private parties under the pretense of mining, before the construction of the dam was begun. This consumed about

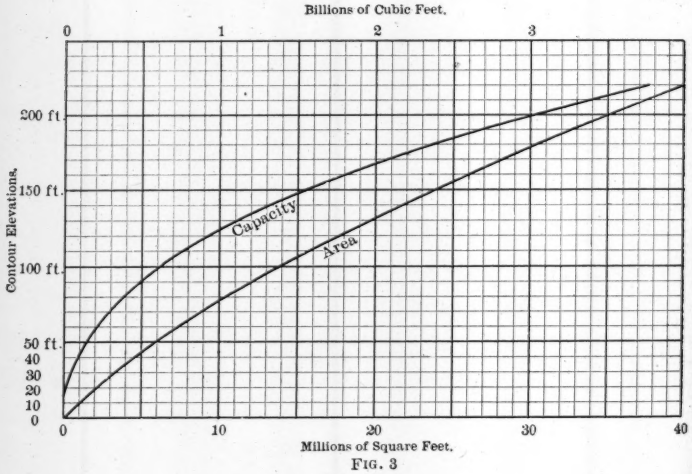


FIG. 3

two years. In the meantime, studies were made by the chief engineer of the various types of dam suitable for the location selected, which was at the narrow cañon just below the junction of the Platte River and Goose Creek (sometimes called Lost Park Creek). See Fig. 2 and also Fig. 1, Plate IV. A section of the dam selected is shown in Fig. 4. It was of the rock-fill type, with a slope of 1½ to 1 on the down-stream side and ½ to 1 on the up-stream side, which latter was to be protected by stone laid by hand and selected from the rock-fill dump. This was to be covered with 12 ins. of concrete to give an even bearing for the steel-plate face, used for making the

PLATE III.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIII, No. 981.
HARRISON AND WOODARD ON
LAKE CHEESMAN DAM AND RESERVOIR.

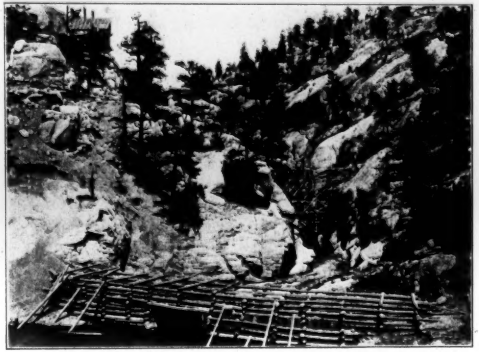


FIG. 1.—VIEW OF CAÑON LOOKING DOWN STREAM.

FIG. 2.—JUNCTION OF PLATTE RIVER AND GOOSE CREEK.

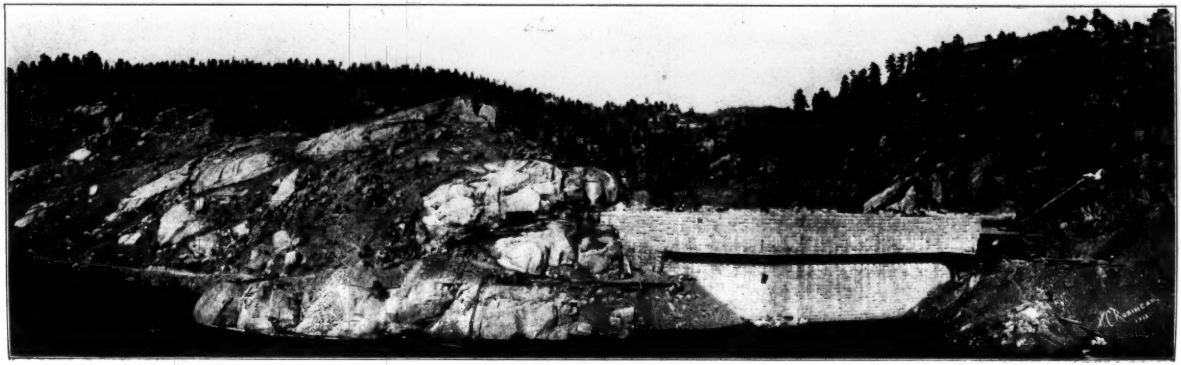


FIG. 3.—GENERAL VIEW OF DAM LOOKING DOWN STREAM.

structure water-tight. The steel face was to be attached to 6-in. I-beams, which were to be anchored into the rock fill with $\frac{5}{8}$-in. rods 5 ft. long. A footing of Portland cement masonry was to be built, into which the steel plates were to be anchored. The top of the dam was to be straight in plan, and the steel face was to be in one plane.

The datum used for the surveys was low water in the river at the site of the dam before any work was done, and it is 6 644 ft. above sea level. This datum has been used throughout the entire work.

It was proposed to draw the water from the reservoir through three tunnels, the intakes of which were at Elevations 10, 60 and 110, as shown on the plan and section, Fig. 1, Plate IV. The lower tunnel was driven and a 42-in. twin valve to regulate the outflow and a guard

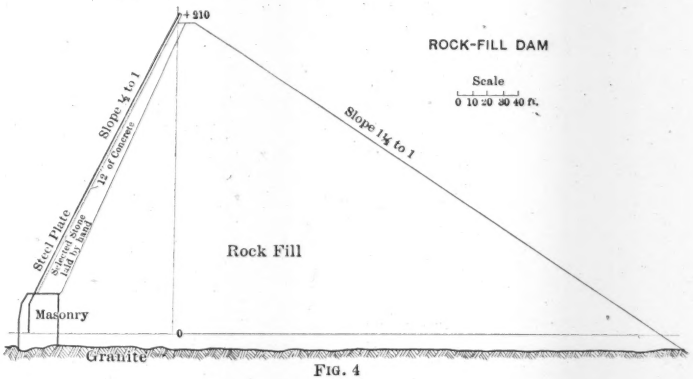


FIG. 4

balance valve at the entrance of the tunnel were placed in position. The flow of the river was then diverted through the tunnel by building a temporary dam across the river just above the site of the proposed permanent dam. Work on the rock-fill dam was then commenced. A bridge was built across the cañon at Elevation 100 (shown in Fig. 2, Plate II), for use in dumping the loose rock into the dam.

The construction of the tunnels and the dam was prosecuted during 1898, 1899, and until May, 1900, at which time the masonry and steel facing were at Elevation 28 and the rock fill behind it at Elevation 54. In the latter part of April, rain, added to the melting snows, caused a flood in the river of greater volume than the discharge capacity of the tunnel. The reservoir was filled rapidly and

on May 3d the flood overtopped the rock fill and washed it away completely, leaving only the masonry and steel facing. A view of the cañon taken a few days after this accident is shown in Fig. 2, Plate II.

The Masonry Dam.

On June 1st, 1900, the writer was appointed Chief Engineer of the South Platte Canal and Reservoir Company and the Denver Union Water Company, with instructions to design and build a dam at this location, with the advice and assistance of L. E. Cooley, M. Am. Soc. C. E., who had been consulting engineer to the company during the previous year.

The important conditions which existed at that time were:

1.—The tunnels at Elevations 10, 60 and 110 had been driven. The balance valve at the entrance of the lower tunnel, and the twin valve in the middle of it, had been permanently set. The two 42-in. single valves for the upper tunnels had been purchased.

2.—The Portland cement masonry, together with the steel plates, had been built, up to Elevation 28, and remained in perfect condition after the flood.

3.—The Government had given five years in which to build the dam, and it was the desire of the management of the company to utilize as much as possible of the work which had already been done and to be as far advanced as possible with the new work at the expiration of the time limit, so as to be in a favorable position to ask for an extension of time in which to complete the work. In no case was the lake to be entirely unwatered.

The type of dam finally determined upon was, in plan and section, substantially that shown in Fig. 1, Plate IV and Fig. 5, and was to be built to Elevation 210, with the spillway at Elevation 200. It was designed as a gravity section, but the configuration of the gorge was such that the natural form of dam to fit the site would be curved in plan. As the full cross-section of a gravity dam could be retained and the arch form used, with no additional masonry, it was decided to curve the up-stream face to a radius of 400 ft. It is built of granite masonry laid in Portland cement mortar. The contract for its construction was let in August, 1900, and the work was begun in September. After the dam had been built to an elevation of about 70,

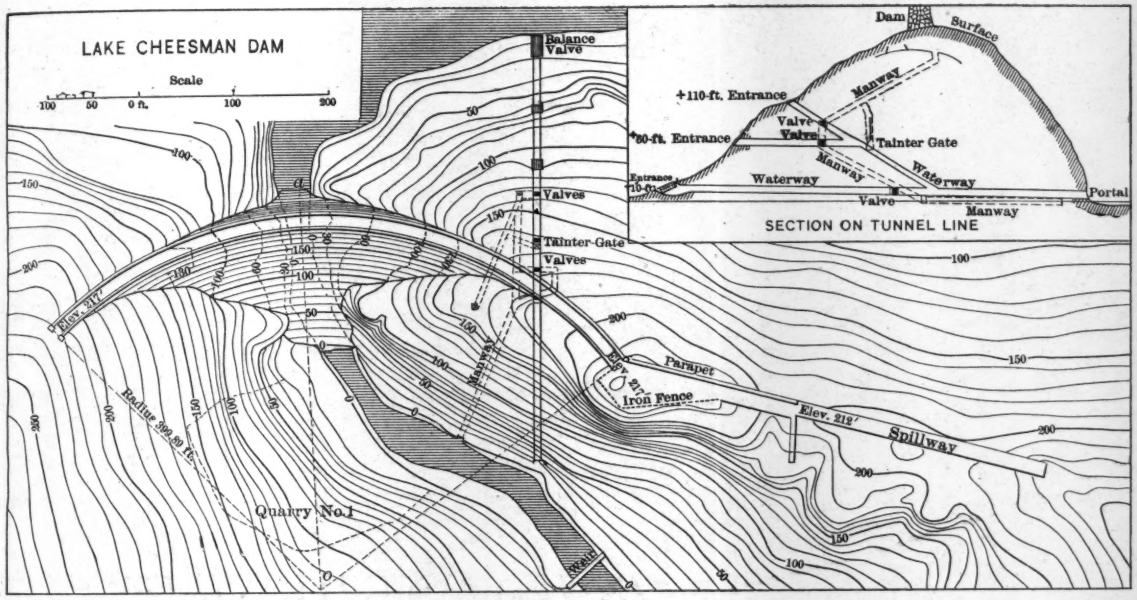
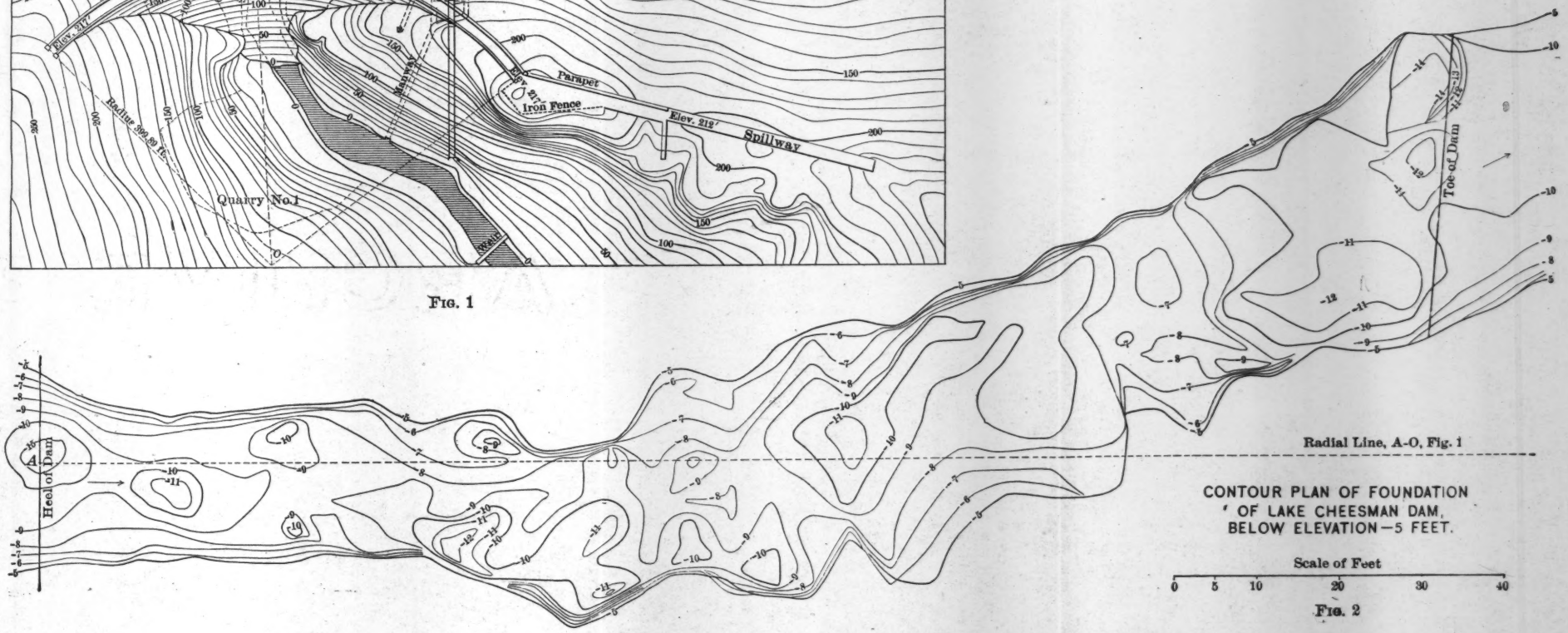


FIG. 1



CONTOUR PLAN OF FOUNDATION
OF LAKE CHEESMAN DAM,
BELOW ELEVATION—5 FEET.

Scale of Feet
0 5 10 20 30 40

FIG. 2

the advisability of carrying it to a greater height than Elevation 210 was considered, with the view of increasing the storage capacity of the reservoir as much as possible, consistent with safety. Taking into consideration the contour of the cañon below Elevation 90, the arch form of dam, and the excellent stone available for its construc-

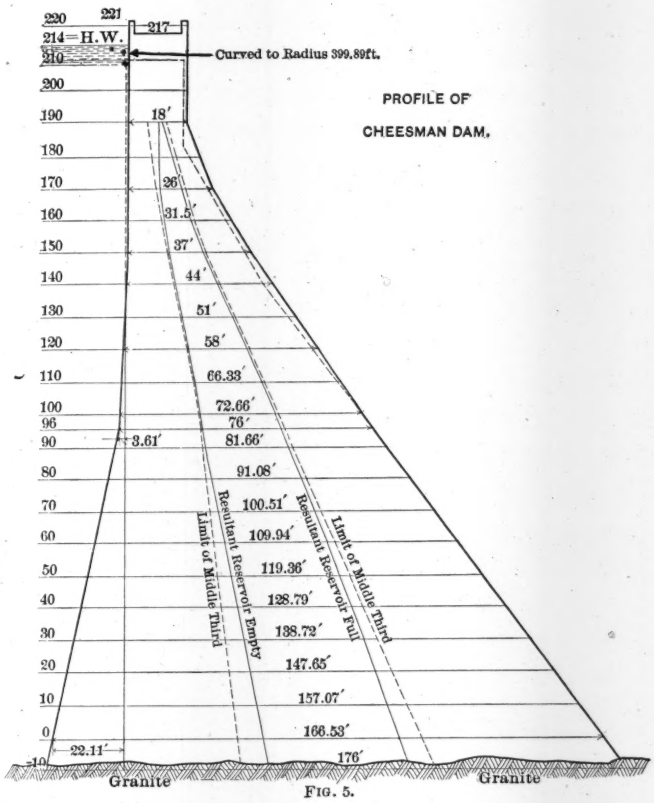


FIG. 5.

tion, it was thought that this could be done, and the problem worked out by the time Elevation 100 was reached with the construction. This elevation was therefore fixed as the point where the change of section should begin. Fig. 2, Plate V, is a view of the cañon with the bottom of the trestle on the dam at about Elevation 100, and

the contours in Fig. 1, Plate IV, give a fair outline of the dam at that elevation. A profile across the cañon is shown in Fig. 7. The question of determining the elevation to which the dam could be built with safety and the section to be adopted presented important and difficult problems. For the solution of them, and for the consideration of the sufficiency of the devices for regulating the outflow from the reservoir, an additional consulting engineer, Alfred Noble, Past-President, Am. Soc. C. E., was appointed.

After trying several sections, that shown in Fig. 5, with the spillway fixed at Elevation 212, was finally adopted. It was designed as a gravity section, but the arch form gives it additional strength. Silas H. Woodard, Assoc. M. Am. Soc. C. E., under the direction of the consulting engineers, made an analysis of the stresses in the dam which is of special interest on account of taking the arch form into consideration. His discussion of this subject is presented by him as a part of this paper.

In computing the weight of the dam, the specific gravity of the masonry is taken as 2.5, or 156.25 lbs. per cubic foot, but actual determinations of the specific gravity of the mortar and the stone (both being dry) in the proportions used in the dam, give a weight of 158 lbs. per cubic foot of masonry. The part of the dam built previous to 1902 was composed of 74% of stone and 26% of mortar.

In computing the stresses in the dam, no account is taken of the ice thrust, for two reasons: (1) Because it is not probable that the reservoir will ever be full during the winter season, and the heavier section of the dam at the water level is more than ample to resist this thrust; and (2) because the point of rock projecting into the reservoir in front of the dam will protect it in such a way as to make this thrust small. This point of rock is shown in Fig. 2 by the contours, and in Fig. 2, Plate V, in the background over the top of the dam. The high-water line is about 10 ft. vertically below the timber-line shown in Fig. 2, Plate V.

Foundation.—The foundation of the dam is the solid granite rock at the bottom of the cañon across which it is built. The widths of this cañon are as follows: At the bottom, about 30 ft.; at Elevation 30, about 40 ft.; at Elevation 90, about 130 ft., on the down-stream face; and at Elevation 217, about 710 ft. (see Fig. 2, Plate V). The elevation of the bed-rock along the axis of the stream is about — 10 ft., but one pot-hole extended down to Elevation — 15 ft. In fact, the entire bot-

FIG. 1.—TOP OF DAM AT ELEVATION 131.

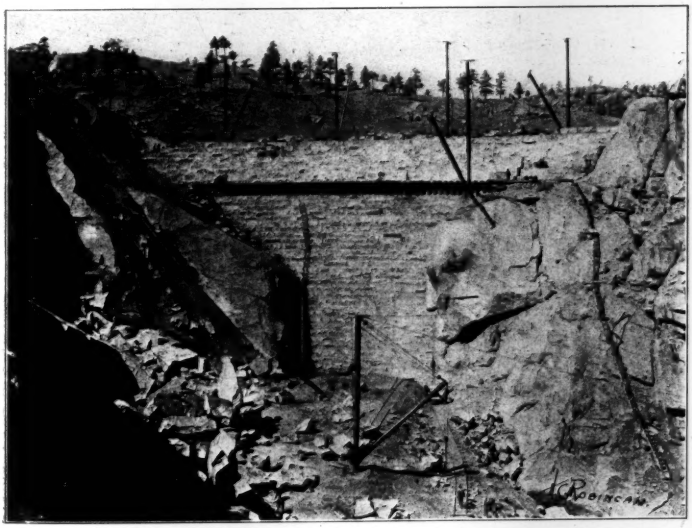


FIG. 2.—DOWN-STREAM FACE OF DAM. TOP OF MASONRY AT ELEVATION 131.

tom was a series of pot-holes eroded in the solid granite and varying in depth from 1 ft. to 6 ft. Similar pot-holes also existed in the walls of the cañon, from the foundation up as high as Elevation 50; some of these, on the south side of the cañon, can be seen in Fig. 1, Plate III. Overlying the bed-rock were boulders and coarse gravel to a depth of about 8 ft. These were removed and the rock was washed clean before the masonry was laid. No unsound rock or crevices were found below Elevation 30, but, above this elevation, some broken and unsound rock was found, and all of it was removed before laying the masonry, thus giving a foundation on solid granite for the entire base of the dam. Generally, the excavation for the foundation consisted of removing large boulders and a few feet of disintegrated rock, but at the south end of the dam and above Elevation 130 there was encountered a large pocket of rock so badly broken up that it was not suitable for a good foundation. Smaller pockets of unsound rock were encountered on both the south and north sides of the cañon. The contour of the foundation of the dam as actually built upon is shown, from Elevation 0 to Elevation 130, by the dotted lines in Fig. 1, Plate IV, and below Elevation — 5 in Fig. 2, Plate IV.

Height of Dam and Depth of Water.—As shown in Fig. 2, Plate V, the lowest part of the foundation along the bed of the river from the heel to the toe of the dam will average about Elevation — 10 ft. The height of the dam from this plane to the roadway on top, at Elevation + 217 ft., is 227 ft. But the extreme low point of the foundation is at Elevation — 15 ft. and the top of the parapet wall is + 221 ft., giving a maximum height of 236 ft. The thickness of the base at Elevation — 10 is 176 ft., and at Elevation 190 it is 18 ft., which thickness is carried uniformly to the top of the structure. Since the material overlying the bed-rock on the up-stream side of the dam is open gravel and boulders, the water pressure against the dam may fairly be assumed to begin at Elevation — 10, and if 2 ft. of water is flowing over the spillway its surface will be at Elevation + 214, making the pressure against the dam that due to a depth of 224 ft. This exceeds very greatly the depth of water against any other dam yet built. There is practically no back pressure from water on the down-stream face of the dam.

Materials Used and Methods of Construction.

The cement used was Portland, and was furnished by the company, free of cost to the contractor, in a warehouse about 4 000 ft. north and

west of the dam. As the needs of the work demanded, it was issued to the contractor, who transported it to the work on scows or wagons, and was responsible for its protection and safekeeping.

It was purchased under the following specifications:

Chemical.—It shall contain neither free lime nor free magnesia, and not more than 2% of sulphate of lime, and the magnesia and oxide of iron shall not be excessive. It shall not be over-burned or under-burned.

Fineness.—Not more than 7% shall be retained on a No. 100 sieve, and not more than 25% on a No. 200 sieve.

Setting.—It shall not take its initial set in less than 45 minutes nor more than 3 hours, nor set hard in less than 3 hours nor more than 8 hours.

Soundness.—It shall show soundness by both the cold-water test and the hot-water test.

Strength.—The neat cement shall develop a tensile strength of 150 lbs. per square inch in one day, and 600 lbs. in 7 days, and show an increase of 15% in the next 21 days. When mixed with sand, as required for mortar, it shall develop a reasonable proportion of the strength of neat cement. Both neat cement and mortar shall show a reasonable increase in strength with age beyond 28 days.

The cement was tested in Denver and then shipped by railroad to Buffalo, a distance of 40 miles. From there it was hauled by teams over a mountain road to the dam, a distance of about 23 miles. In addition to the tests made at Denver, frequent tests for tensile strength of mortar, taken from the mortar boxes on the work, have been made, since 1902, by the resident engineer. Altogether, about 80 000 bbls. of cement will be required. Of this amount, 6 000 bbls., used in 1900, were of the Wolverine Brand. All the cement used since then has been the Iola Portland cement, made at Iola, Kansas.

The sand is furnished by the contractor and is secured from a gulch about 5 000 ft. north and west of the dam. It is run through a $\frac{3}{8}$-in. screen and washed, after which it is taken to the dam on scows or wagons. As actually used in the work, it contains, as determined by several analyses, less than one-tenth of 1% of volatile and organic matter, and has about 31% voids.

The mortar for laying the stone on the up-stream face and at the bottom and ends of the dam, adjoining the solid rock, is mixed in

FIG. 1.—ENTRANCE TO LOWEST TUNNEL.

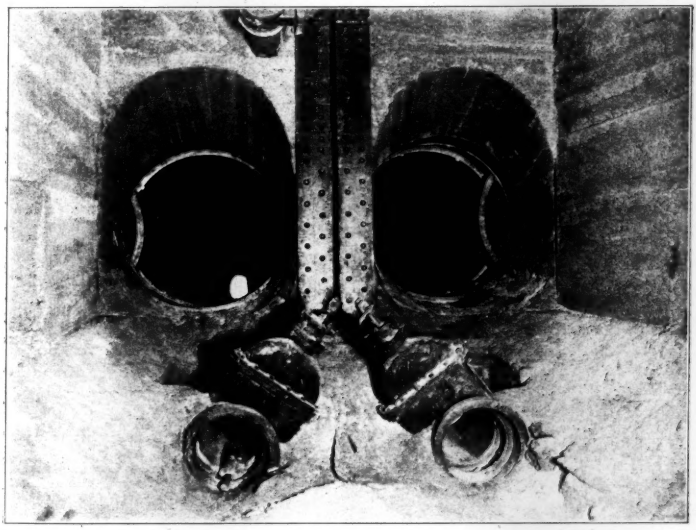


FIG. 2.—TWIN VALVES IN TUNNEL.

proportions of 2 parts of sand to 1 part of cement by volume, and 95 lbs. of cement are taken as 1 cu. ft.; the mortar for the rest of the masonry is mixed in the proportion of 2½ to 1. The sand and cement are mixed dry, the proper quantity of water is added, and then the mixing is continued. A batch mixer, manufactured by the Iroquois Iron Works, of Buffalo, N. Y., is used for this purpose, and gives excellent results. It is located near the end of the dam, and the mortar, after being mixed, is dumped into cars which are moved along a track built on brackets on the up-stream face of the dam, as shown in Fig. 3, Plate III. From this track the mortar is hoisted to the work on the dam by the several derricks used for setting stone, thus avoiding any interference with the other derricks, and ensuring the delivery of the mortar at the point where it is to be used.

The stone used is a good gray granite. The stone for the up-stream face is obtained from Quarry No. 2 (Fig. 2) distant about 2 000 ft. north and west from the dam. The stones are rough-pointed, so as to be laid with horizontal beds and vertical joints, and the outside edges are made to conform to the curvature of the dam. No attempt is made to dress the exposed faces of the stones. The specifications required them to be laid with ¾-in. joints, but this requirement was modified, with the approval of the consulting engineers, so as to permit 1-in. joints, in the belief that the thicker bed of mortar would be more nearly water-tight. The specifications also allowed these stones to be laid in broken courses, but the contractors preferred to have them of a uniform thickness, and all the face stones are uniformly 2 ft. thick. All joints are raked out to a depth of 1½ ins. and pointed with 2 to 1 mortar. The specifications required that one-fourth of the face area should be headers, from 4 to 6 ft. long, not less than 2 ft. wide, and evenly distributed through the wall. The stretchers were to be not less than 3 ft. and not more than 7 ft. long, with a width not less than one and one-half times the thickness. The stones for the down-stream face are not dressed, but are large and well shaped, and of a thickness approximating 2 ft. They are selected and laid in steps, so as to give a good appearance for rough work. The rubble for the interior of the dam is of good-sized stones, well shaped, and laid so as to break joints and bond in all directions. In laying the stones a full bed of soft mortar is required, and, in filling in between the large stones, it is required that the space shall first be

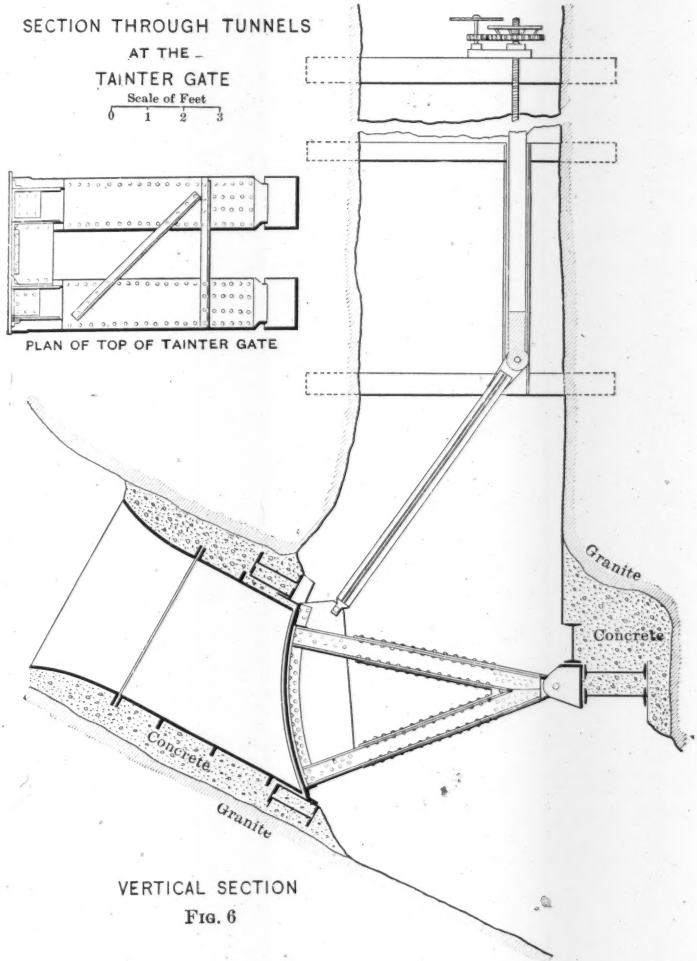
filled with mortar and the smaller stones then worked down into it. This is considered a very important feature in building water-tight masonry, and, so far, it has proved effective in this case. During the past summer the water stood in the reservoir at about Elevation 97, and there was no evidence of a leak or even sweating on the lower face of the dam. The stone for the down-stream face and the interior of the dam is secured from Quarry No. 1 (Fig. 1, Plate IV), situated just below the dam. It is transported from the quarry to the toe of the dam on small cars, from which it is hoisted to cars running on a trestle built along the face of the dam. See Fig. 2, Plate V. The building derricks take the stone from these cars as needed for use in the work. This arrangement for distributing the stone avoids any interference of one derrick with another.

The stones for the interior of the dam are set so as to bond vertically as well as horizontally, and special care is taken not to level up the work at any point throughout the thickness of the wall. This, in addition to giving the proper bond, makes it very difficult for water to enter into or seep through the dam. The photograph, Fig. 1, Plate V, taken when the top of the masonry was at Elevation 131, is a fair illustration of how the stones were disposed, and also shows the shape of the stones used in the work. In the quarry, large masses of stone are loosened with powder and are then reduced by plug-and-feather to the proper size for building into the dam.

Devices for Regulating the Outflow.

As a part of the plan for building the rock-fill dam, already mentioned, it was proposed to draw the water from the reservoir by tunnels driven through the granite mountain, as shown by the solid lines in the plan and the "Section on Tunnel Line," Fig. 1, Plate IV.

Before the writer's connection with the work, these tunnels had been driven, and a balance valve, protected by a grating, had been set permanently at the intake of the lower tunnel, as shown in Fig. 1, Plate VI. In addition to this, a 42-in. twin valve had been set at about the middle of this tunnel, a view of which is shown in Fig. 2, Plate VI. The balance valve was designed to be operated by a hydraulic cylinder, the water supply for which was to be carried in



VERTICAL SECTION

FIG. 6

pipes along the bottom of the tunnel from its portal to the valve, and a waste pipe was provided in the same manner. The twin valves were to be operated by hydraulic cylinders supplied with water in a similar manner. The valves in the 60-ft. and 110-ft. tunnels had been provided, but were not set.

The fact that no provision had been made to give access to the working parts of these valves when water was being drawn out through the tunnels seemed to be a serious objection, and was met by driving a separate "Manway tunnel," as shown by the dotted lines in Fig. 1, Plate IV, thus at all times giving free access to the working parts of the valves and also providing a safe means of piping the water for operating them. It was also thought advisable to provide means for creating a back pressure on the valves in the 60-ft. and 110-ft. tunnels, so as to make them easily operated when the reservoir was full. For this purpose, a "Tainter Gate," to be operated by hand, was designed and placed just below the junction of the two tunnels, as shown in Fig. 1, Plate IV, and Fig. 6. It is probable that this gate may prove to be the best one to operate, under ordinary conditions, for drawing water out of the reservoir.

Spillway.

After the reservoir is filled, the flood water will discharge from it into the river below the dam, over a spillway about 300 ft. long, at Elevation 212. The south end of the spillway is about 200 ft. north of the north end of the dam, as shown in Fig. 1, Plate IV, and Fig. 3, Plate III, and is in a natural saddle in the rock ridge. The surface of the rock varies from about Elevation 195 at the south end to Elevation 225 at the north end. Where the rock is above Elevation 212, it is to be excavated to that level and the lower portion is to be built up with rubble masonry, similar to that in the main dam, to Elevation 212 and a top width of 14 ft.

The maximum flood recorded in the river occurred in June, 1900, and was 1 945 cu. ft. per second. Several times this volume of water would discharge over the spillway before the level of the lake would reach the top of the dam. In addition to this the storage capacity of the reservoir will act as an equalizer, and more than 1 000 cu. ft. per second can be drawn off through the tunnels.

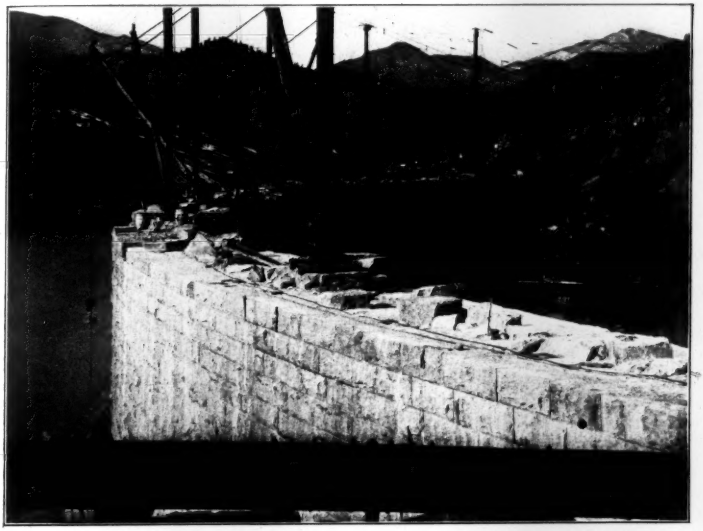


FIG. 1.—VIEW OF TOP OF DAM LOOKING NORTH. ELEVATION 210.

FIG. 2.—VIEW OF TOP OF DAM LOOKING NORTH. ELEVATION 217.

QUANTITIES.

The principal quantities of work involved in the construction of the reservoir, dam and appurtenances are as follows:

1.—Excavating 26 000 cu. yds. of earth and rock for foundations.

2.—Driving about 1 300 ft. of water and manway tunnels, and setting five valves in these tunnels, and gratings at the intakes.

3.—Building 103 000 cu. yds. of masonry.

4.—Clearing about 1 000 acres of timber and brush from the site of the reservoir.

The striking features of the location for this reservoir and dam are:

1.—A rocky, mountainous water-shed, producing but little vegetation and containing a small population, which makes it practically free from pollution.

2.—A reservoir site, composed of granite rock or granitic sand, of such contour as to give great depth and capacity with a small surface area.

3.—A site for the dam which affords good, solid foundations and a narrow cañon for the lower part of the structure, thus making the quantity of masonry required comparatively small. To this is added a good rock foundation for a spillway at a convenient location.

The writer's connection with this work ended on May 24th, 1902, at which time the masonry of the dam was completed to about Elevation 134, and the contracts then let provided for building it to Elevation 210. The reservoir site had been cleared, and the tunnels were nearly completed. No contracts had been let for that part of the masonry in the dam above Elevation 210, nor for the excavation and masonry between the north end of the dam and the north end of the spillway. The general plan of this part of the work was as shown in Fig. 1, Plate IV, but the details, as actually built, have been designed by Alexander E. Kastl, M. Am. Soc. C. E., who has been Chief Engineer of the company since May 24th, 1902.

On January 1st, 1904, the main dam was completed for nearly its entire length to Elevation 217, and it is expected that the entire work will be completed before July 1st, 1904. The masonry and excavation have been done under contract by The Geddis and Seerie Stone Company, and all other work has been done by the company by day's labor.

In addition to the Chief and Consulting Engineers already mentioned, the following have been immediately connected with the work:

James E. Maloney, Resident Engineer to April 1st, 1902; Frank C. Horn, M. Am. Soc. C. E., Resident Engineer since April 1st, 1902; John A. Runner, Assistant Engineer; M. A. McGraw and C. C. Murphy, Inspectors.

Part II.

ANALYSIS OF STRESSES IN LAKE CHEESMAN DAM.

By Silas H. Woodard, Assoc. M. Am. Soc. C. E.

Most of the dams which have been built during the last fifty years have been straight in plan, depending upon their weight for stability.

In a few cases, where the dams have closed very narrow valleys, they have been curved up stream, and designed as arches to transmit all the thrust of the water to the sides of the valley. There is a third type, still fewer in number, located in moderately narrow valleys, which have been made the full, or nearly full, cross-section which would be required for a gravity dam, and, besides, have been curved in plan.

What is practically a standard method of analysis of stresses in a gravity dam, based upon the usual assumption of uniformly varying stress, has been developed. This method is simple, and may be found, with variations, in detail in the textbooks and works upon masonry, and has been used in the design of practically all modern dams.

The analysis of stresses of the purely arch dams has been made after still more common methods in use for all masonry arches. Many objections to this method of treatment have been raised. The objectors have argued that the fact that the horizontal arch is held rigidly at the base of the dam upsets the whole assumption that the dam acts simply as an arch. If the arched form is maintained and the thickness is increased, so that the structure has considerable strength as a gravity section, this argument has even greater force.

The Lake Cheesman Dam belongs to this last class, of combination gravity and arch dams, and the analysis of stresses was made in three ways.

In the first its cross-section was examined as a gravity section by the methods usually applied to gravity dams.

In the second the stresses were computed when the dam was considered as a simple horizontal arch. This was done only incidentally and with no idea that the assumptions at all approximated the actual conditions, but it shows what the strength of the arch would be if it could be developed. The dam could not fail without developing this strength.

The third analysis takes account of the combined action of the gravity section and the horizontal arch.

There is nothing novel in the first two treatments, and the only excuse for this paper is found in the third. The method here used is not itself novel, but it has not been usually applied to masonry dams.

The analysis of the stresses in every engineering structure, with a very few exceptions, is based upon the assumption that all particles lying in a plane before stressing will lie in a plane after stressing, and that stress is proportional to strain, or, as it is commonly expressed, the stress is uniformly varying along any plane.

This law of distribution of stress has been tested as to its correctness, when applied to the common cases of steel, timber and small masonry construction, by the most refined laboratory experiments, and also by the success of countless structures for which it has formed the basis of design. Yet it must have its limit of practical application, and great care should be exercised in extending it to new fields.

It is an open question whether or not the application of the law to a masonry dam, 150 ft. thick at its base, is an unwarranted assumption. It will probably remain so until some one expends the time and money to make the delicate measurements necessary to solve the problem.

While the applicability of the law to masonry dams is not as well demonstrated as to an I-beam, for example, there are very good theoretical reasons for believing that, even if the law is not rigidly correct when applied to masonry dams, its error is on the safe side. Besides this, the law has been applied to the design of masonry dams for half a century, and, as far as known, every successful dam satisfies the requirements of stability which are founded upon the law, and all failures may be explained either by a disregard of those requirements or by other obvious reasons.

Granting the correctness of the above law, the stresses for any loading of the structure may be computed by any one of a great variety of methods. By comparing these stresses with known safe limits, the safety of the structure may be judged.

In the following computation of the stresses in the dam as a gravity section, the prism of masonry included between two parallel, transverse, vertical planes, 1 ft. apart, is considered to resist the water pressure against its up-stream face.

This prism of masonry is considered to be divided into layers by imaginary horizontal joints or planes 10 ft. apart, as indicated in Fig. 5. The condition of stress is then computed at each one of these joints.

Let b = the breadth of the horizontal joint;

W = the weight of the masonry above the joint, in terms of the weight of a cubic foot of masonry = the sectional area above the joint (weight of 1 cu. ft. of masonry = 156.25 lbs.);

P = the horizontal thrust of the water pressure acting above the joint, in terms of the weight of a cubic foot of masonry $= \frac{d^2}{2} \times \frac{62.5}{156.25} = \frac{d^2}{5}$;

d = the depth of the joint below high water = 214 = the elevation of the joint;

S_1 = the batter of the up-stream face of the dam in the 10 ft. above the joint;

S_2 = the batter of the down-stream face of the dam in the 10 ft. above the joint;

M = the moment of all the masonry above the joint about its up-stream edge;

n = the distance from the up-stream side of the dam to the resultant for reservoir empty $= \frac{M}{W}$;

a = the angle which the resultant for reservoir full makes with the vertical;

tan. $a = \frac{P}{W}$ = the coefficient of friction required to prevent sliding;

v = the distance between the positions of the resultants for reservoir full and reservoir empty $= \frac{P}{W}\frac{d}{3} = \frac{d^3}{15\ W}$;

$n - \frac{b}{3}$ = the distance which the resultant for reservoir empty falls inside the middle third;

$\frac{2\,b}{3} - (n + v)$ = the distance which the resultant for reservoir full falls inside the middle third;

f_1 = the vertical intensity of pressure, in pounds per square foot, at the up-stream edge of the joint, for reservoir empty $= \frac{312.5\ W}{b^2}(2\,b - 3\,n)$;

f_2 = the vertical intensity of pressure, in pounds per square foot, at the down-stream edge of the joint, for reservoir empty $= \frac{312.5\ W\ (3\,n - b)}{b^2}$;

f'_1 = the vertical intensity of pressure, in pounds per square foot, at the up-stream edge of the joint, for reservoir full $= \frac{312.5\ W\,(2\,b - 3\ (n + v))}{b^2}$;

f'_2 = the vertical intensity of pressure, in pounds per square foot, at the down-stream edge of the joint, for reservoir full $= \frac{312.5\ W\,(3\ (n + v) - b)}{b^2}$.

The results of the computation at each horizontal joint are given in Table 2.

It is usually required that friction alone should be sufficient to prevent sliding along any horizontal plane. This requires that the resultant at any horizontal joint shall make an angle with the vertical the tangent of which is less than the coefficient of friction.

In the Lake Cheesman Dam there is nothing like a horizontal joint, for great care has been taken to make vertical bonds, and the masonry can be relied upon to develop great strength in shear. However, the required coefficients of friction to prevent sliding have been worked out in the table.

Two columns are also given showing the amount that the resultants, for reservoir full and reservoir empty, fall inside the middle third of each horizontal joint.

The positions of the resultants are plotted in Fig. 5.

This treatment of the curved dam must be considered, at best, as an approximation, and has force only if it be assumed that of two dams of the same cross-section, one of which is curved, with rigid abutments, and the other straight, the curved one will be at least as strong as the other.

TABLE 2.—COMPUTATION OF STRESSES IN DAM AT LAKE CHEESMAN.

Elevation of joint.	Breadth of joint. b	Total masonry above joint. W	Batter of up-stream face in 10 ft. above joint. S_1	Batter of down-stream face in 10 ft. above joint. S_2	Moment of masonry about up-stream edge. M	Distance from up-stream edge to resultant, reservoir empty. N	Distance between results, reservoir full and reservoir empty. V	Tan. α	$n-\frac{b}{3}$	$\frac{2}{3}b-(n+v)$	Reservoir Empty.		Reservoir Full.	
											Stress on up-stream edge.	Stress on down-stream edge.	Stress on up-stream edge.	Stress on down-stream edge.
190.....	18	504	0	.....	4 536	9	1.96	0.25	3	1.04	4 380	4 380	1 520	7 230
180.....	22	704	0	4	6 543	9.29	3.72	0.33	1.96	1.66	7 330	2 670	2 260	7 740
170.....	26	944	0	4	9 430	10	6.01	0.41	1.33	1.32	9 600	1 550	1 540	9 600
160.....	31.5	1 232	0.14	5.36	13 727	11.14	8.52	0.47	0.64	1.33	11 480	740	1 560	10 670
150.....	37	1 574	0.14	5.36	19 801	12.58	11.10	0.52	0.25	0.98	13 030	270	1 064	12 230
140.....	44	1 979	0.6	6.4	29 085	14.69	13.64	0.55	0.03	1.00	14 030	20	1 150	12 900
130.....	51	2 454	0.6	6.4	41 724	17.00	16.10	0.58	0.00	0.90	15 030	0	800	14 240
120.....	58	2 999	0.6	6.4	58 228	19.41	18.46	0.59	0.08	0.89	16 100	60	670	15 500
110.....	65.33	3 616	0.6	6.73	79 245	21.91	20.75	0.60	0.13	0.90	17 200	100	710	16 600
100.....	72.66	4 306	0.6	6.73	105 836	24.45	22.96	0.60	0.28	1.03	18 350	180	790	17 730
90.....	81.66	5 072	1.486	7.514	142 598	28.11	25.06	0.61	0.89	1.27	18 780	630	910	18 510
80.....	91.08	5 936	1.927	7.5	190 522	32.09	27.02	0.61	1.73	1.61	19 200	1 160	1 060	19 280
70.....	100.51	6 894	1.927	7.5	248 790	36.09	28.87	0.60	2.59	2.10	19 780	1 650	1 310	20 130
60.....	109.94	7 946	1.927	7.5	318 470	40.08	30.64	0.60	3.43	2.58	20 490	2 120	1 590	21 020
50.....	119.36	9 092	1.927	7.5	400 627	44.06	32.34	0.59	4.27	3.18	21 240	2 550	1 900	21 690
40.....	128.79	10 330	1.927	7.5	496 339	48.05	33.99	0.59	5.12	3.82	22 080	2 990	2 230	22 830
30.....	138.22	11 668	1.927	7.5	606 679	52.00	35.68	0.58	5.93	4.46	23 010	3 400	2 560	23 850
20.....	147.65	13 098	1.927	7.5	732 713	55.94	37.16	0.57	6.72	5.33	23 940	3 790	3 000	24 780
10.....	157.07	14 621	1.927	7.5	875 505	59.86	38.71	0.57	7.50	6.14	24 910	4 080	3 410	25 660
0.....	166.53	16 239	1.927	7.5	1 036 134	63.80	40.23	0.56	8.30	6.97	25 930	4 560	3 830	26 660
−10.....	175.93	17 951	1.927	7.5	1 215 679	67.96	41.74	0.56	9.32	7.58	26 830	5 070	4 130	27 770

It was assumed that a prism of masonry cut by parallel planes resisted the water pressure upon 1 ft. length of dam. In reality, a wedge of masonry cut by radial planes must resist this water pressure. At the toe this wedge would be only six-tenths of its thickness at the upstream face.

There is little doubt that the curved form, of the same cross-section, is as strong as the straight, and the whole question might be left here with confidence in the stability of the dam. But this is only another way of saying that the arch must carry its part of the load, and relying on it to do so without any measure of how much this part is.

It was thought very desirable to gain a better idea of what part of the load is resisted by the arch and what part by the gravity section. To accomplish this, the following method of analysis was used:

It is evident that the load will be divided between the arch and the gravity dam in proportion to their relative rigidities, and it is upon this that the analysis is based.

The gravity section might have been treated as a vertical cantilever beam projecting upward from the foundation.

If the stresses in this beam had been computed by the usual method of treating beams, and these stresses corrected for the weight of masonry lying above the section considered, the same results would have been obtained as those given in Table 2. In the following analysis this method was used.

Any structure which resists force must suffer deformation, and, if the material is not stressed beyond its elastic limit, the amount of deformation is proportional to the force which produces it. In other words, the amount of deformation is a measure of the forces which cause it.

The structure may be built so that one simple system is deformed under load, as in the case of a simple girder or a simple arch; in that case it is evident that the one system which is deformed is sustaining the whole load. If, however, two or more systems are deformed, which would be the case if, for example, the middle of a beam rested upon an arch or a strut, then it follows that the load is divided between the two systems in proportion to their rigidity. This is the case of the curved masonry dam. The thrust of the water on the upstream face causes the dam to deflect down stream. This means that the horizontal arch is distorted and therefore resists a certain amount of thrust which it transmits to the sides of the valley, and the dam

considered as a gravity section, or, as pointed out before, as a cantilever beam projecting upward from the foundation, is also deformed, and therefore resists a certain amount of the load, which it conducts to the base of the dam. The problem is to determine how much of the load goes each way, and then to test each system for its ability to carry its load with safety.

The method to be pursued is to divide the dam above Elevation 55, which, for reasons to be given later, will be considered to be its base, into five arch rings by horizontal planes at Elevations 100, 130, 160 and 190. An expression will be developed which will give the deflection of each of these arches, in terms of its properties and the load which it resists.

Another set of expressions will be developed which will give the deflections of the points on the dam at the elevations of the middle of the arch rings, if the dam is considered as a vertical beam, in terms of the properties of the vertical beam and the loads which it resists.

It is useless to attempt an exact solution of the problem, as the outlines of the structure are not sufficiently regular, and to enter into too great refinements would complicate the solution, which, at best, is somewhat involved. However, an exact solution is not necessary, nor especially desirable. If whatever approximations are made are upon the safe side, and results can be gotten which are known to be within limits which are not too wide, the solution is nearly as satisfactory and quite as valuable, as a test of the safety of the structure, as though it were exact.

In the Lake Cheesman Dam, if it be considered that the dam, acting as an arch, carries all the load, the stresses would not be excessive. This, in itself, is a very good guaranty of the stability of the dam, for if it should fail utterly as a gravity section it could never be carried away until it had failed as an arch, and it seems safe against such a failure. It follows, then, that if either feature of the dam is in danger it must be the gravity section, and, therefore, the safe side, in an assumption for the calculation, is the one which throws the greater load upon the vertical cantilever or gravity section.

There are two assumptions that it is necessary to make, to simplify the work.

First, it is assumed that the line of thrust in the arch coincides with its center line throughout its length. In other words, the effect of the fixed abutments is neglected. This is on the safe side, for the

effect of the fixed abutments is to stiffen the arch and make it take more of the load, leaving less for the gravity section.

Second, it is assumed that the bottom of the dam follows the dotted lines *a b c* in Fig. 7. This, again, is on the safe side, for, if the

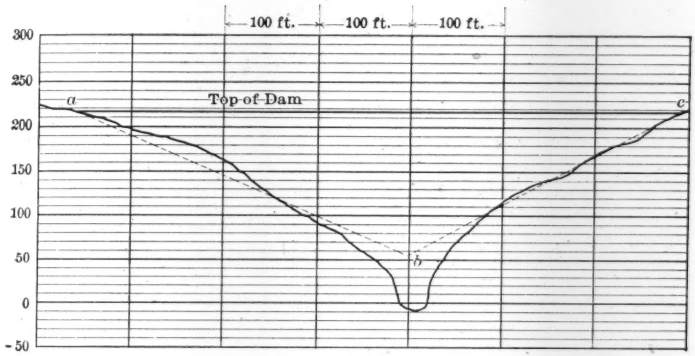


FIG. 7

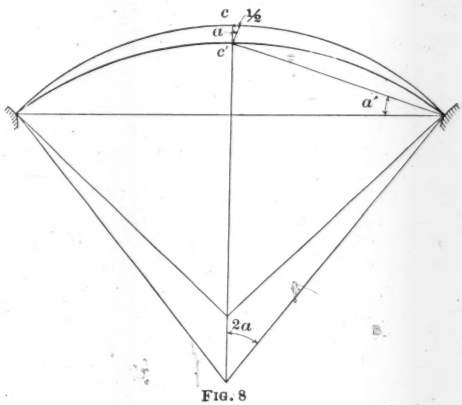


FIG. 8

bottom of the dam at its middle were at *b*, Fig. 7, the vertical cantilever would be shorter, and therefore stiffer, than it actually is, and, therefore, it would take more load than it actually does. Or, looked at in another way, the assumption places the bed-rock at *b*, and implies

that it is perfectly rigid. As a matter of fact, there is a mass of 65 ft. of masonry below b which is stressed at its down-stream edge to, say, 30 000 lbs. per square foot, and must compress, allowing the dam to deflect at the middle still more than has been assumed, and throwing more of the load upon the arches and less upon the gravity section than has been assumed. Let Fig. 10 represent a section of the dam at its middle, or at b, Fig. 7, showing the section of the horizontal arches. Let the arches be numbered from the top down, and let all subscripts refer to this numbering. Consider a vertical beam fixed at b, Fig. 7, and consisting of the masonry enclosed between two vertical radial planes 1.002 ft. apart at the up-stream top edge of the dam. Consider this beam to rest against the horizontal arches at Elevations 205, 175, 145, 115 and 85, and suppose the water pressure to be concentrated at these points. These pressures, in terms of the weight of a cubic foot of masonry, are:

$$P_1 = 115$$
$$P_2 = 468$$
$$P_3 = 831$$
$$P_4 = 1\,197$$
$$P_5 = 1\,573$$

Let the loads upon the vertical beams at Elevations 205, 175, 145, 115 and 85 be X_1, X_2, X_3, X_4, and X_5. It will be assumed, for the moment, that the arches have a uniform loading throughout their length. This loading must then be $P - X$. Later, this assumption will be shown to be warranted.

If, as has been assumed, the arch has hinged ends, its deflection will be as indicated in Fig. 8, that is, the deflection is uniformly increasing from the ends toward the middle.

Let A = the sectional area of an arch ring;
R = the radius of its up-stream edge;
E = the modulus of elasticity of the masonry;
a = one-fourth the angle which the arch subtends;
T = the thrust in the arch ring;
L = the length of the center line of the arch ring;
l = the shortening of the arch ring under compression;
c = the initial position of the crown of the arch (see Fig. 8);
c' = the position of the crown after deflection;

D = the deflection of the crown of the arch;

$$D = \frac{l}{2} \text{ cot. } a;$$

but $$l = \frac{T\,L}{A\,E} = \frac{(P - X)\,R\,L}{A\,E}.$$

Then $$D = \frac{(P - X)\,R\,L \text{ cot. } a}{2\,A\,E} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(I)}$$

The following are the values of R, L, A and cot. a, for the several arches:

$R_1 = 400$	$L_1 = 580$	$A_1 = 504$	cot. $a_1 = 2.5$
$R_2 = 400$	$L_2 = 400$	$A_2 = 728$	cot. $a_2 = 3.7$
$R_3 = 400$	$L_3 = 270$	$A_3 = 1\,222$	cot. $a_3 = 4.7$
$R_4 = 402.4$	$L_4 = 200$	$A_4 = 1\,851$	cot. $a_4 = 7.1$
$R_5 = 405.7$	$L_5 = 130$	$A_5 = 2\,589$	cot. $a_5 = 11.4$

Before going farther, the stress in the arches, if the structure had no resistance whatever as a gravity dam, will be investigated. We now have all the elements for substituting in the approximate formula,

$$F = \frac{P\,R}{A},$$

for stress in an arch.

For 1st arch, $F = \dfrac{115 \times 400 \times 156.25}{504} = 14\,000$ lbs. per square foot.

For 2d arch, $F = \dfrac{468 \times 400 \times 156.25}{728} = 40\,000$ lbs. per square foot.

For 3d arch, $F = \dfrac{831 \times 400 \times 156.25}{1\,222} = 42\,500$ lbs. per square foot.

For 4th arch, $F = \dfrac{1\,197 \times 402.4 \times 156.25}{1\,851} = 41\,200$ lbs. per square foot.

For 5th arch, $F = \dfrac{1\,573 \times 405.7 \times 156.25}{2\,589} = 38\,500$ lbs. per square foot.

By this it is seen that the arches would not be over-stressed if the dam had no resistance as a gravity section.

By substituting the values of R, L, H and cot. a, in Equation I, the following set of equations results:

$$E\,D_1 = 575.4\,(P_1 - X_1) = 66\,170 - 575.4\,X_1$$
$$E\,D_2 = 406.6\,(P_2 - X_2) = 190\,300 - 406.6\,X_2$$
$$E\,D_3 = 207.7\,(P_3 - X_3) = 172\,600 - 207.7\,X_3$$
$$E\,D_4 = 154.5\,(P_4 - X_4) = 184\,950 - 154.5\,X_4$$
$$E\,D_5 = 116.1\,(P_5 - X_5) = 182\,600 - 116.1\,X_5$$

These are E times the deflections of the crown of the several arches, in terms of the loads on the vertical beam.

These same deflections will now be expressed in terms of the properties of the vertical beam and its load.

The deflection of the dam, considered as a vertical beam, is expressed by the formula:

$$D_n = \int_0^H \frac{M\, d\, M}{E\, I\, d\, X_n}\, d\, y \qquad \text{(II)}$$

in which M = the bending moment at any point;

I = the moment of inertia of the beam;

E = the modulus of elasticity;

H = the height of the dam = the total length of the beam;

D_n = the deflection of any loaded point, from the position of the beam without loads. D must be measured in the direction of the action of the load X_n.

For the reason that this formula does not appear very generally in textbooks, it will be demonstrated here.

Let any beam be acted upon by loads X_1, X_2.....X_n at points D_1, D_2.....D_n.

Let the movement of these points under load be D_1, D_2.....D_n. In passing from one condition of loading to another, work is done. The total work of all the external forces acting upon the beam is $-W = \sum \frac{P\, D}{2}$.

By the doctrine of the conservation of energy, this must be equalled by the internal work done in distorting the beam. This internal work is done in lengthening and compressing the material, and is stored as potential energy in the material, to be given back if the forces are removed and the beam springs back to its initial position.

In Fig. 9:

Let $c\ c$ be an infinitesimal portion of length, $d\ y$, of the neutral axis of the beam under stress;

$n\ n'$ and $m\ m'$ be normal sections;

i be the angle, $n'\ c\ r'$;

z be the distance from the neutral axis to any point, q, on the section, $n\ n'$;

f be the stress at the point, q;

$Z_1 = c\ n$;

$Z_2 = c\ n'$.

Before the beam was stressed, $n\ n'$ was parallel to $m\ m'$, or in a position, $r\ r'$.

The average force acting upon a unit area at q is $\frac{f}{2}$, and the distance through which it acts is $z\ i$.

FIG. 9

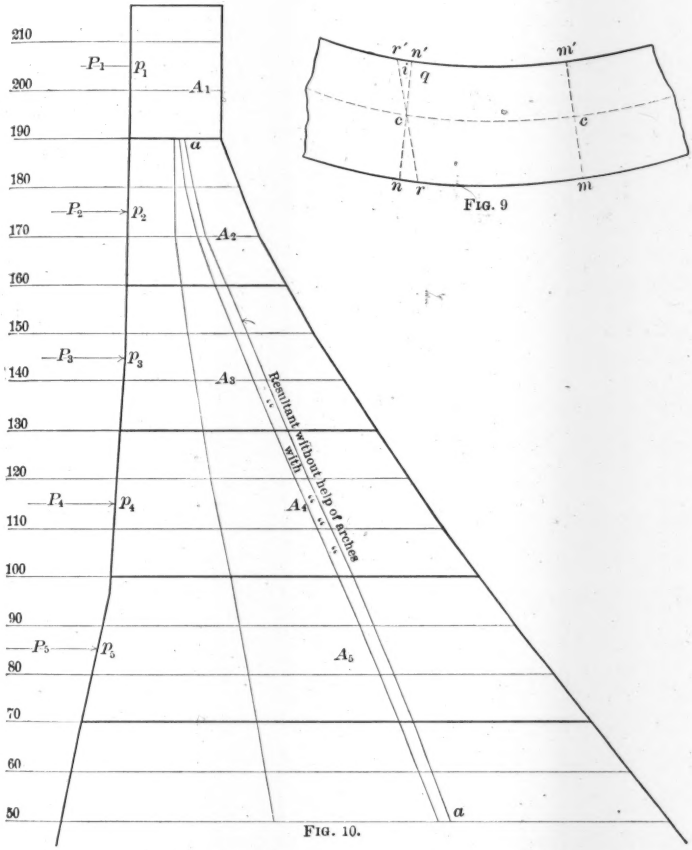


FIG. 10.

Then, neglecting the distortion caused by shear, which is practically zero, the work done on an area, a, at q is: $w = \frac{f\ a\ z\ i}{2}$.

The work upon the whole prism, $n\ n'\ m'\ m$, is:

$$W = \sum_{Z_1}^{Z_2} \frac{f\ a\ z\ i}{2}.$$

The total internal work in the whole beam is:

$$W = \sum_{o}^{H} \sum_{-Z_1}^{Z_2} \frac{f\,a\,z\,i}{2}.$$

The total external work done upon the beam is:

$$W = \frac{P_1\,D_1 + P_2\,D_2 - - - - P_n\,D_n}{2}.$$

Equating external with internal work, and clearing of fractions, we have:

$$P_1\,D_1 + P_2\,D_2 - - - P_n\,D_n = \sum_{o}^{H} \sum_{-Z_1}^{Z_2} f\,a\,z\,i.$$

Differentiating this equation with respect to P_n, we have:

$$D_n = \sum_{o}^{H} \sum_{-Z_1}^{Z_2} \frac{df}{dP_n}\,a\,z\,i.$$

But, $$f = \frac{M\,z}{I}.$$

Then, $$\frac{df}{dP_n} = \frac{d\,M\,z}{d\,P_n\,I};$$

and $$z\,i = \frac{f\,dy}{E} = \frac{M\,z\,d\,y}{E\,I}.$$

Substituting the values of $\frac{df}{dP_n}$ and $z\,i$, we have:

$$D_n = \int_{o}^{H} \sum_{-Z_1}^{Z_2} \frac{d\,M}{d\,P_n}\,\frac{z}{I}\,\frac{a\,M\,z\,d\,y}{E\,I} =$$

$$\int_{o}^{H} \frac{d\,M}{d\,P_n}\,\frac{M}{E\,I^2} \sum_{-Z_1}^{Z_2} a\,z^2\,d\,y;$$

but $$\sum_{-Z_1}^{Z_2} a\,z^2 = I.$$

We therefore have

$$D_n = \int_{o}^{H} \frac{d\,M}{d\,P_n}\,\frac{M}{E\,I}\,d\,y \ldots\ldots\ldots\ldots \text{Q. E. D.}$$

Let M_1 be the bending moment at any point between p_1 and p_2, Fig. 10, and M_2 between p_2 and p_3, and so on for M_3, M_4 and M_5.

Let I_1 be the average moment of inertia between p_1 and p_2, I_2 the same between p_2 and p_3, etc.

Let y be the distance measured downward from any load point toward the next.

Then $M_1 = y\,X_1$, and $\frac{d\,M_1}{d\,X_1} = y$

$$\int_o^a \frac{d\,M_1\,M_1}{E\,I_1\,d\,X_1}\,d\,y = \frac{a^3}{6\,E\,I_1}\,(2\,X_1)$$

$$M_2 = (y + a)\,X_1 + y\,X_2$$

$$\frac{d\,M_2}{d\,X_1} = y + a, \qquad \frac{d\,M_2}{d\,X_2} = y$$

$$\frac{d\,M_2\,M_2}{d\,X_1} = (y^2 + 2\,a\,y + a^2)\,X_1 + (y^2 + a\,y)\,X_2$$

$$\int_o^a \frac{d\,M_2\,M_2}{E\,I_2\,d\,X_1}\,d\,y = \frac{1}{E\,I_2}$$

$$\left[\left(\frac{a^3}{3} + a^3 + a^3\right)X_1 + \left(\frac{a^3}{3} + \frac{a^3}{2}\right)X_2\right] =$$

$$\frac{a^3}{6\,E\,I_2}\,(14\,X_1 + 5\,X_2)$$

$$\frac{d\,M_2\,M_2}{d\,X_2} = (y^2 + a\,y)\,X_1 + y^2\,X_2$$

$$\int_o^a \frac{d\,M_2\,M_2\,d\,y}{E\,I_2\,d\,X_2} = \left[\left(\frac{a^3}{3} + \frac{a^3}{2}\right)X_1 + \frac{a^3}{3}\,X_2\right]\frac{1}{E\,I_2} =$$

$$\frac{a^3}{6\,E\,I_2}\,(5\,X_1 + 2\,X_2)$$

$$M_3 = (2\,a + y)\,X_1 + (a + y)\,X_2 + y\,X_3$$

$$\frac{d\,M_3}{d\,X_1} = (2\,a + y), \quad \frac{d\,M_3}{d\,X_2} = a + y, \quad \frac{d\,M_3}{d\,X_3} = y$$

$$\frac{d\,M_3\,M_3}{d\,X_1} = (4\,a^2 + 4\,a\,y + y^2)\,X_1 + (2\,a^2 + 3\,a\,y + y^2)\,X_2 +$$
$$(2\,a\,y + y^2)\,X_3$$

$$\int_o^a \frac{d\,M_3\,M_3\,d\,y}{d\,X_1} = \frac{1}{E\,I_3}$$

$$\left[\left(4\,a^3 + 2\,a^3 + \frac{a^3}{3}\right)X_1 + \left(2\,a^3 + \frac{3\,a^3}{2} + \frac{a^3}{3}\right)X_2 + \left(a^3 + \frac{a^3}{3}\right)X_3\right] =$$

$$\frac{a^3}{6\,E\,I_3}\,(38\,X_1 + 23\,X_2 + 8\,X_3)$$

$$\frac{d\,M_3\,M_3}{d\,X_2} = (2\,a^2 + 3\,a\,y + y^2)\,X_1 + (a^2 + 2\,a\,y + y^2)\,X_2 + (a\,y + y^2)\,X_3$$

$$\int_o^a \frac{d\,M_3\,M_3\,d\,y}{E\,I_3\,d\,X_2} = \frac{1}{E\,I_3}$$

$$\left[\left(2\,a^3 + \frac{3\,a^3}{2} + \frac{a^3}{3}\right)X_1 + \left(a^3 + a^3 + \frac{a^3}{3}\right)X_2 + \left(\frac{a^3}{2} + \frac{a^3}{3}\right)X_3\right] = \frac{a^3}{6\,E\,I_3}(23\,X_1 + 14\,X_2 + 5\,X_3)$$

$$\frac{d\,M_3\,M_3}{d\,X_3} = (2\,a\,y + y^2)\,X_1 + (a\,y + y^2)\,X_2 + y^2\,X_3$$

$$\int_o^a \frac{d\,M_3\,M_3}{E\,I_3\,d\,X_3}\,d\,y = \frac{1}{E\,I_3}$$

$$\left[\left(a^3 + \frac{a^3}{3}\right)X_1 + \left(\frac{a^3}{2} + \frac{a^3}{3}\right)X_2 + \frac{a^3}{3}\,X_3\right] = \frac{a^3}{6\,E\,I_3}\,(8\,X_1 + 5\,X_2 + 2\,X_3)$$

$$M_4 = (3a + y)\,X_1 + (2a + y)\,X_2 + (a + y)X_3 + y\,X_4$$

$$\frac{d\,M_4}{d\,X_1} = (3\,a + y),\ \frac{d\,M_4}{d\,X_2} = (2\,a + y),\ \frac{d\,M_4}{d\,X_3} = a + y,\ \frac{d\,M_4}{d\,X_4} =$$

$$\frac{d\,M_4\,M_4}{d\,X_1} = (9\,a^2 + 6\,a\,y + y^2)\,X_1 + (6\,a^2 + 5\,a\,y + y^2)\,X_2 + (3\,a^2 + 4\,a\,y + y^2)\,X_3 + (3\,a\,y + y^2)\,X_4$$

$$\int_o^a \frac{d\,M_4\,M_4}{E\,I_4\,d\,X_1}\,d\,y = \frac{a^3}{6\,E\,I}\,(74\,X_1 + 53\,X_2 + 32\,X_3 + 11\,X_4)$$

$$\frac{d\,M_4\,M_4}{d\,X_2} = (6\,a^2 + 5\,a\,y + y^2)\,X_1 + (4\,a^2 + 4\,a\,y + y^2)\,X_2 + (2\,a^2 + 3\,a\,y + y^2)\,X_3 + (2\,a\,y + y^2)\,X_4$$

$$\int_o^a \frac{d\,M_4\,M_4}{E\,I_4\,d\,X_2}\,d\,y = \frac{a^3}{6\,E\,I_4}\,(53\,X_1 + 38\,X_2 + 23\,X_3 + 8\,X_4)$$

$$\frac{d\,M_4\,M_4}{d\,X_3} = (3\,a^2 + 4\,a\,y + y^2)\,X_1 + (2\,a^2 + 3\,a\,y + y^2)\,X_2 + (a^2 + 2\,a\,y + y^2)\,X_3 + (a\,y + y^2)\,X_4$$

$$\int_o^a \frac{d\,M_4\,M_4}{E\,I_4\,d\,X_3} = \frac{a^3}{6\,E\,I_4}\,(32\,X_1 + 23\,X_2 + 14\,X_3 + 5\,X_4)$$

$$\frac{d\,M_4\,M_4}{d\,X_4} = (3\,a\,y + y^2)\,X_1 + (2\,a\,y + y^2)\,X_2 + (a\,y + y^2)\,X_3 + y^2\,X_4$$

$$\int_o^a \frac{d\,M_4\,M_4}{E\,I_4\,d\,X_4} = \frac{a^3}{6\,E\,I_4}(11\,X_1 + 8\,X_2 + 5\,X_3 + 2\,X_4)$$

$$M_5 = (4\,a + y)\,X_1 + (3\,a + y)\,X_2 + (2\,a + y)\,X_3 + (a + y)\,X_4 + y\,X_5$$

$$\frac{d\,M_5}{d\,X_1} = (4\,a + y),\quad \frac{d\,M_5}{d\,X_2} = (3\,a + y),\quad \frac{d\,M_5}{d\,X_3} = (2\,a + y),\quad \frac{d\,M_5}{d\,X_4} = (a + y),\quad \frac{d\,M_5}{d\,X_5} = y$$

$$\frac{d\,M_5\,M_5}{d\,X_1} = (16\,a^2 + 8\,a\,y + y^2)\,X_1 + (12\,a^2 + 7\,a\,y + y^2)\,X_2 + (8\,a^2 + 6\,a\,y + y^2)\,X_3 + (4\,a^2 + 5\,a\,y + y^2)\,X_4 + (4\,a\,y + y^2)\,X_5$$

$$\int_o^a \frac{d\,M_5\,M_5}{E\,I_5\,d\,X_1}\,d\,y = \frac{a^3}{6\,E\,I_5}(122\,X_1 + 95\,X_2 + 68\,X_3 + 41\,X_4 + 14\,X_5)$$

$$\frac{d\,M_5\,M_5}{d\,X_2} = (12\,a^2 + 7\,a\,y + y^2)\,X_1 + (9\,a^2 + 6\,a\,y + y^2)\,X_2 + (6\,a^2 + 5\,a\,y + y^2)\,X_3 + (3\,a^2 + 4\,a\,y + y^2)\,X_4 + (3\,a\,y + y^2)\,X_5$$

$$\int_o^a \frac{d\,M_5\,M_5}{E\,I_5\,d\,X_2}\,d\,y = \frac{a^3}{6\,E\,I_5}(95\,X_1 + 74\,X_2 + 53\,X_3 + 32\,X_4 + 11\,X_5)$$

$$\frac{d\,M_5\,M_5}{d\,X_3} = (8\,a^2 + 6\,a\,y + y^2)\,X_1 + (6\,a^2 + 5\,a\,y + y^2)\,X_2 + (4\,a^2 + 4\,a\,y + y^2)\,X_3 + (2\,a^2 + 3\,a\,y + y^2)\,X_4 + (2\,a\,y + y^2)\,X_5$$

$$\int_o^a \frac{d\,M_5\,M_5}{E\,I_5\,d\,X_3}\,d\,y = \frac{a^3}{6\,E\,I_5}(68\,X_1 + 53\,X_2 + 38\,X_3 + 23\,X_4 + 8\,X_5)$$

$$\frac{d\,M_5\,M_5}{d\,X_4} = (4\,a^2 + 5\,a\,y + y^2)\,X_1 + (3\,a^2 + 4\,a\,y + y^2)\,X_2 + (2\,a^2 + 3\,a\,y + y^2)\,X_3 + (a^2 + 2\,a\,y + y^2)\,X_4 + (a\,y + y^2)\,X_5$$

$$\int_o^a \frac{d\,M_5\,M_5}{E\,I_5\,d\,X_4}\,d\,y = \frac{a^3}{6\,E\,I_5}(41\,X_1 + 32\,X_2 + 23\,X_3 + 14\,X_4 + 5\,X_5)$$

$$\frac{d\,M_5\,M_5}{d\,X_5} = (4\,a\,y + y^2)\,X_1 + (3\,a\,y + y^2)\,X_2 + (2\,a\,y + y^2)\,X_3 + (a\,y + y^2)\,X_4 + y^2\,X_5$$

$$\int_o^a \frac{d\,M_5\,M_5}{E\,I_5\,d\,X_5}\,d\,y = \frac{a^3}{6\,E\,I_5}$$

$$(14\,X_1 + 11\,X_2 + 8\,X_3 + 5\,X_4 + 2\,X_5)$$

$$I_1 = \quad 540$$
$$I_2 = \; 3\,240$$
$$I_3 = 10\,800$$
$$I_4 = 32\,400$$
$$I_5 = 81\,000$$

To simplify the numerical work, the factor, $36 = C$, is introduced. Then,

$$\frac{a^3\,C}{6\,E\,I_1} = \frac{300}{E}$$
$$\frac{a^3\,C}{6\,E\,I_2} = \frac{50}{E}$$
$$\frac{a^3\,C}{6\,E\,I_3} = \frac{15}{E}$$
$$\frac{a^3\,C}{6\,E\,I_4} = \frac{5}{E}$$
$$\frac{a^3\,C}{6\,E\,I_5} = \frac{2}{E}$$

$$D_1 = \int_o^H \frac{M\,d\,M}{E\,I\,d\,X_1}\,d\,y = \int_o^a \frac{M_1\,d\,M_1}{E\,I_1\,d\,X_1}\,d\,y +$$

$$\int_o^a \frac{M_2\,d\,M_2}{E\,I_2\,d\,X_1}\,d\,y \ldots\ldots\ldots + \int_o^a \frac{M_5\,d\,M_5}{E\,I_5\,d\,X_1}$$

Or, by making the proper substitutions, we get:

$$C\,E\,D_1 = 2\,484\,X_1 + 1\,050\,X_2 + 416\,X_3 + 137\,X_4 + 28\,X_5$$
$$C\,E\,D_2 = 1\,050\,X_1 + \quad 648\,X_2 + 296\,X_3 + 104\,X_4 + 22\,X_5$$
$$C\,E\,D_3 = \quad 416\,X_1 + \quad 296\,X_2 + 176\,X_3 + \quad 71\,X_4 + 16\,X_5$$
$$C\,E\,D_4 = \quad 137\,X_1 + \quad 104\,X_2 + \quad 71\,X_3 + \quad 38\,X_4 + 10\,X_5$$
$$C\,E\,D_5 = \quad 28\,X_1 + \quad 22\,X_2 + \quad 16\,X_3 + \quad 10\,X_4 + \quad 4\,X_5$$

By multiplying the values of $E\,D_1$, $E\,D_2$, etc., which were found from the arch, by the factor, 36, we have:

$$C\,E\,D_1 = 2\,382\,000 - 20\,714\,X_1$$
$$C\,E\,D_2 = 6\,851\,000 - 14\,638\,X_2$$
$$C\,E\,D_3 = 6\,214\,000 - \quad 7\,477\,X_3$$
$$C\,E\,D_4 = 6\,658\,000 - \quad 5\,562\,X_4$$
$$C\,E\,D_5 = 6\,574\,000 - \quad 4\,180\,X_5$$

We have now two sets of expressions for the same thing. By equating these we get the following group of simultaneous equations:

$$0 = -2\,382\,000 + 23\,198\,X_1 + 1\,050\,X_2 + 416\,X_3 + 137\,X_4 + 28\,X_5$$
$$0 = -6\,851\,000 + 1\,050\,X_1 + 15\,286\,X_2 + 269\,X_3 + 104\,X_4 + 22\,X_5$$
$$0 = -6\,214\,000 + 416\,X_1 + 296\,X_2 + 7\,653\,X_3 + 71\,X_4 + 16\,X_5$$
$$0 = -6\,658\,000 + 137\,X_1 + 104\,X_2 + 71\,X_3 + 5\,600\,X_4 + 10\,X_5$$
$$0 = -6\,574\,000 + 28\,X_1 + 22\,X_2 + 16\,X_3 + 10\,X_4 + 4\,184\,X_5$$

Solving these equations we get:

$$X_1 = 61$$
$$X_2 = 420$$
$$X_3 = 780$$
$$X_4 = 1\,167$$
$$X_5 = 1\,570$$

These are the thrusts to be sustained by the gravity section, and may now be used in place of the actual to compute the stresses in the gravity section.

The loads on the arches are:

$$P_1 - X_1 = 55$$
$$P_2 - X_2 = 48$$
$$P_3 - X_3 = 51$$
$$P_4 - X_4 = 30$$
$$P_5 - X_5 = 3$$

By comparing these with the loading which goes to the gravity section, it is seen that the arch carried nearly half the load at the top, but only about 6% half way down, and practically none at the bottom. However, any assistance near the top of a gravity dam is worth many times the same amount near the bottom. Table 3 gives the computation of stresses in the trapezoidal section included between two radial planes for both the case of reservoir empty and that of reservoir full. For the horizontal thrusts of water pressure, X_1, X_2, etc., just found, were used down to Elevation 70. Below this point the actual water pressures were used. In this table,

b = the breadth of the joint;
C_1 = the width of the up-stream edge of the joint;
C_2 = " " " down-stream edge of the joint;
r = " radius " " " " "
s_1 = " batter " up-stream face in the 10 ft. next above the joint;

s_2 = the batter of the down-stream face in the 10 ft. next above the joint;

a = the area of the horizontal joint = $\frac{(C_1 + C_2)\ b}{2}$;

W = the weight of all masonry above the joint, in terms of the weight of a cubic foot of masonry;

g = the distance of the center of gravity of the joint from its up-stream edge = $\frac{b\,(C_1 - 2C_2)}{3\,(C_1 + C_2)}$;

M = the moment of all the masonry above a joint about its up-stream edge;

n = the distance from the up-stream edge of a joint to the resultant for reservoir empty = $\frac{M}{W}$;

t = the actual thrust of the water against the 10-ft. layer next above a joint, in terms of the weight of a cubic foot of masonry;

X = the thrust of the water against the 10-ft. layer next above a joint which is carried by the gravity section (see the foregoing computation of the effect of arch action);

M_1 = the moment about the joint of all these loadings (X) which act above it;

v = the distance between the resultants for reservoir full and reservoir empty = $\frac{M_1}{W}$;

I = the moment of inertia of the joint about its center of gravity;

f_1 = the vertical stress, in pounds per square foot, at the up-stream edge when the reservoir is empty, =

$$\frac{156.25\ W}{a} + \frac{156.25\ W\,(g - n)\,g}{I};$$

f_2 = the vertical stress, in pounds per square foot, at the down-stream edge when the reservoir is empty, =

$$\frac{156.25\ W}{a} + \frac{156.25\ (g - n)\ (b - g)}{I};$$

f'_1 = the vertical stress, in pounds per square foot, at the up-stream edge of the joint when the reservoir is full, =

$$\frac{156.25\ W}{a} - \frac{156.25\ W\,(n + v - g)\,g}{I}.$$

f'_2 = vertical stress, in pounds per square foot, at the downstream edge of the joint when the reservoir is full, =

$$\frac{156.25\ W}{a} + \frac{156.25\ W\ (n + v - g)\ (b - g)}{I}$$

In discussing Table 3 it should be noted that with this treatment of the subject there is no question of whether or not the resultants fall within the middle third. That condition applies only when the horizontal sections are rectangular. However, to show the effect of the arch action upon the position of the resultants, they are plotted in Fig. 10. The line a—a shows the position of the resultants for reservoir full if the gravity section receives no assistance from the arch form. The adjacent line shows the position of the resultants for reservoir full when the gravity section is assisted by the arches.

The last four columns of Table 3 are of principal interest. They show no tension and no compression greater than is safe for the excellent masonry used in the dam. At the toe the stress is about 35 000 lbs. per square foot. There are two reasons why this stress can probably never be developed. The lower 50 ft. of the dam is in a deep, narrow, crooked cañon, and if at the toe a horizontal straight line is drawn at right angles to the dam it will soon run into the solid granite rock of the cañon. This is shown very well in Fig. 2, Plate IV. It is also probable that the masonry of the dam itself will bridge across the narrow cañon, delivering much of its weight to the side walls.

The position of the resultants for reservoir full and reservoir empty, as shown by Table 3, is shown in Fig. 10.

The line a–a shows the position of the resultants for reservoir full if the gravity section receives no assistance from the arches.

The assumption was made, in the beginning of the analysis, that the arches were loaded uniformly, and that their deflection increased uniformly from zero at the abutments to a maximum at the middle. We have now to show that the assumption was warranted.

By substituting 61 for X_1 and 36 for C, in the equation giving the deflection of the top arch, we get

$$D_1 = \frac{31\ 100}{E}.$$

This is the deflection at the middle of the dam. By the assumption, the deflections, at points on the arch, one-fifth, two-fifths, three-

TABLE 3.

Elevation of joint.	Breadth of joint.	Width of up-stream edge of joint.	Width of down-stream edge of joint.	Radius of down-stream edge of joint.	Batter of up-stream face, in 10 ft. above the joint.	Batter of down-stream face, in 10 ft. above the joint.	Area of horizontal joint.	Total weight of masonry above the joint.	Distance of center of gravity of joint from up-stream edge.	Total moment of masonry above up-stream edge.	Distance from up-stream edge to resultant, reservoir empty.	Actual thrust of water on 10-ft. layer above joint.	Thrust of water on 10-ft. layer carried by gravity section.	Moment of all X's above joint.	Distance between results, reservoir empty and reservoir full.	Moment of inertia of joint.	Reservoir Empty.		Reservoir Full.	
																	Stress on up-stream edge.	Stress on down-stream edge.	Stress on up-stream edge.	Stress on down-stream edge.
	b	C_1	C_2	r	S_1	S_2	a	W	g_1	M	n	t	X	M_1	v	I	f_1	f_2	f'_1	f'_2
190..	18	1.002	0.956	381.89	0	....	17.62	493	8.93	4 401	8.93	115	61	488	0.99	484	4 380	4 380	2 970	5 800
180..	22	1.002	0.946	377.89	0.	4.00	21.43	689	10.89	6 344	9.22	116	100	1 598	2.32	865	7 280	2 720	4 140	5 920
170..	26	1.002	0.936	373.89	0	4.00	25.20	922	12.85	9 114	9.90	156	140	3 908	4.23	1 423	9 550	1 790	4 490	7 064
160..	31.5	1.002	0.923	368.53	0.14	5.36	30.32	1 199	15.53	13 212	11.02	196.1	180	7 818	6.53	2 518	11 020	1 300	3 500	8 940
150..	37	1.002	0.910	363.17	0.14	5.36	35.37	1 528	18.19	18 952	12.40	226.9	220	13 728	8.99	4 083	12 200	870	2 730	10 880
140..	44	1.004	0.894	356.77	0.6	6.40	41.76	1 914	21.58	27 728	14.48	276.9	260	22 038	11.52	6 728	13 570	490	2 520	11 970
130..	51	1.005	0.878	350.37	0.6	6.40	48.02	2 363	24.93	39 464	16.70	317.4	300	33 148	14.04	10 373	14 680	360	2 230	13 380
120..	58	1.007	0.862	343.97	0.6	6.40	54.20	2 874	28.25	54 529	18.97	358.1	348	47 498	16.53	15 180	15 800	340	1 890	15 020
110..	65.33	1.008	0.845	337.24	0.6	6.73	60.53	3 447	31.71	73 732	21.39	398.9	389	65 530	19.02	21 438	16 930	370	1 780	16 440
100..	72.66	1.01	0.828	330.51	0.6	6.73	66.77	4 084	34.92	97 211	23.80	440.1	430	87 663	21.47	30 320	17 580	680	1 800	17 940
90..	81.66	1.013	0.809	323.	1.486	7.514	74.39	4 789	39.31	130 021	27.15	481.5	480	114 343	23.88	41 154	18 630	820	1 570	19 200
80..	91.08	1.018	0.79	315.5	1.927	7.5	82.34	5 573	43.63	172 513	30.96	524.0	523	146 038	26.21	56 661	18 970	1 440	1 400	20 550
70..	100.51	1.023	0.771	308.	1.927	7.5	90.16	6 436	47.95	223 590	34.75	567.4	566	183 178	28.47	75 084	18 550	1 950	1 270	21 980
60..	109.94	1.028	0.753	300.5	1.927	7.5	97.90	7 376	52.15	283 976	38.61	611.2	611	226 203	30.67	97 440	20 060	2 580	1 140	23 550
50..	119.36	1.033	0.734	293.	1.927	7.5	105.45	8 393	56.32	354 319	42.22	655.5	655	275 558	32.83	124 580	20 740	3 140	1 280	24 940
40..	128.79	1.037	0.715	285.5	1.927	7.5	112.83	9 484	60.38	435 094	45.88	699.3	699	331 683	34.97	155 270	21 440	3 710	1 290	26 550
30..	138.22	1.042	0.696	278.	1.927	7.5	120.11	10 649	64.52	527 242	49.52	744.3	744	395 023	37.10	188 790	22 340	4 150	1 250	28 250
20..	147.65	1.047	0.678	270.5	1.927	7.5	127.35	11 886	68.57	631 304	53.12	789.7	790	466 033	39.22	226 940	23 220	4 620	1 280	30 000
10..	157.07	1.052	0.659	263.	1.927	7.5	134.42	13 195	72.53	747 830	56.68	835.4	835	545 168	41.31	271 840	24 020	4 960	1 300	31 720
0..	166.5	1.057	0.640	255.5	1.927	7.5	141.27	14 573	76.43	877 255	60.20	881.6	882	632 888	43.43	320 300	24 900	5 780	1 340	33 540
—10..	175.93	1.062	0.621	248.	1.927	7.5	148.04	16 020	80.28	1 020 165	63.67	928.1	928	729 658	45.54	373 800	25 800	6 330	1 340	35 470

fifths and four-fifths of the distance from the middle toward the abutment, will be $\frac{24\,800}{E}$, $\frac{18\,600}{E}$, $\frac{12\,400}{E}$, and $\frac{6\,200}{E}$, respectively.

By substituting the values of X_1, X_2, X_3, X_4 and X_5, in the equations for the deflection of the vertical beam or gravity section, we get $D_1 = \frac{31\,100}{E}$, which checks the values just found for the middle of the top arch.

If the base of the dam follows the line *a–b–c*, Fig. 7, then at the points one-fifth, two-fifths, three-fifths and four-fifths of the distance from middle toward the abutments, the base will be at Elevations 85, 115, 145 and 175. Computing the deflection of the tops of the gravity sections loaded with X_1, X_2, etc., the bases of which are at these elevations, the deflections are:

$$\frac{25\,000}{E}, \quad \frac{16\,000}{E}, \quad \frac{8\,000}{E} \text{ and } \frac{800}{E}.$$

These agree very well with those above, except at the three-fifths and four-fifths points, where the deflection of the beam is much less. This means that the gravity section may take more load than X_1 and X_2, but it is abundantly strong to carry all the water pressure and more, and it seems that the assumption was warranted, especially as no account has been taken of the stiffness of the fixed abutment of the arch, which would tend to make the deflection of the arch less near the abutments.

It will be noticed that it has not been necessary to assign an absolute value to E; the foregoing analysis would have little value had it been necessary to do so, for the modulus of elasticity of masonry is, not only different for each piece of work, but is different for different ages of the same mass.

It has been assumed, simply, that the modulus is the same, both for horizontal and vertical directions. There appears to be no reason why this should not be so.

The value of E probably lies somewhere between 2 000 000 and 4 000 000 lbs. per square inch, or say 3 000 000. This, in terms of the weight of a cubic foot of masonry and a square foot, is 2 764 000. If this value be substituted in Equation $D_1 = \frac{31\,100}{E}$, we get $D_1 = 0.011$ ft. for the deflection at the top and middle point of the dam.

Thus far, only the forces of water pressure and gravity have been considered as acting upon the dam. Besides the stresses induced by these forces, there will be internal stresses caused by the change of volume of the mortar in setting, the building of portions of the dam more rapidly than others, and certain portions in cold weather and others in warm weather.

Of these stresses, probably those due to temperature changes are much the greatest. The dam has been built nearly continuously, winter and summer, so that it is practically impossible to set the range of temperature which will produce stress. When the reservoir is full, about 30° Fahr. is as great a change of temperature as can be expected. This would tend to make the arch in summer carry more load than has been attributed to it by the analysis, and less in winter, supposing that the dam is always at a uniform temperature. Should the dam be cooler on the down-stream face than on the other, which will nearly always be the case in winter, the effect would be to throw more load on the arch, thus neutralizing the effect of the winter condition of the arch. Likewise, a warm down-stream face would neutralize the effect of the summer lengthening of the arch.

If the dam should stand through a cold winter with no water against its up-stream face, cracks might be expected in the upper part, which was built for the most part in summer. This is the common experience with straight dams.

The curved form, having greater flexibility, would tend to reduce the number and size of the cracks, and, in a dam like the Bear Valley Dam, the arch would probably be flexible enough to prevent them entirely. This cannot be expected in the Lake Cheesman Dam, but, no doubt, it will prevent any temperature cracks when the reservoir is full.

A theoretical analysis of stress is always open to more or less doubt, and probably the best test of a design is a comparison with precedents whenever it is possible.

Fig. 11 shows superimposed profiles of several dams which are comparable with the Lake Cheesman Dam, as follows:

The Villar Dam, in Spain, built in 1870 to 1878, of rubble masonry, and curved to a radius of 440 ft.

The Chatrain or Tache Dam, in France, built in 1888 to 1892, of rubble masonry, and curved to a radius of 1 312 ft.

The Furens Dam, in France, built in 1862 to 1866, of rubble masonry, and curved to a radius of 828 ft.

The Ban Dam, in France, built in 1867 to 1870, of rubble masonry, and curved in plan.

The Periyar Dam, in India, built in 1888 to 1895, of concrete, and straight in plan.

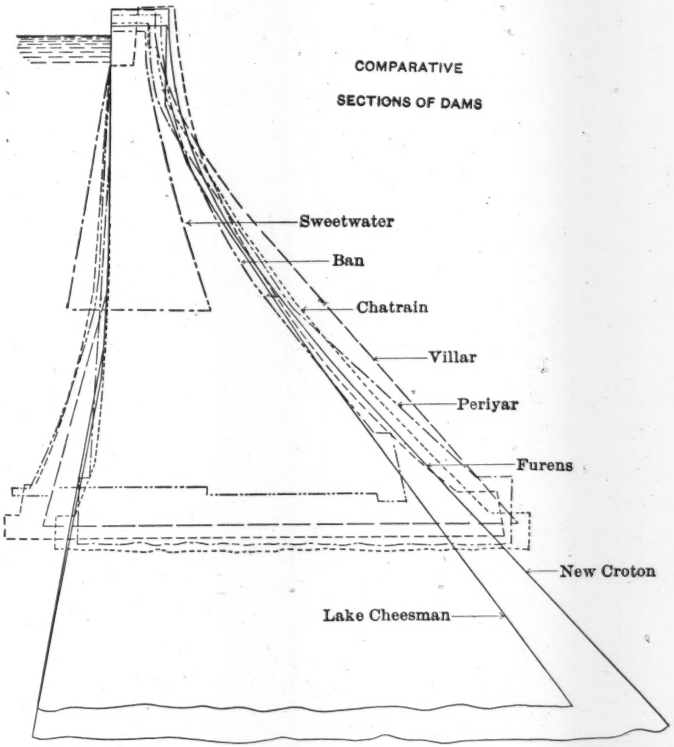


FIG. 11

The Sweetwater Dam, in California, built in 1887 and 1888, of rubble masonry, and curved to a radius of 222 ft.

The profiles shown in broken lines in Fig. 11 are all of dams which have been built and have been subjected to pressure for some years, and are the best precedents existing. The five first named are

the highest dams in the world which have been subject to test. All except the Periyar are curved in plan. The Sweetwater, however, is the only one in which the designers relied on the arched form for strength. In this case the arched form is the main reliance, although the dam has considerable resistance as a gravity section. The adopted profile of the Lake Cheesman Dam is shown by a full line. Compared with all but the Sweetwater Dam, it seems rather slender. There are, however, three conditions which offset this:

The lower portion shown in the profile is little more than a plug of a narrow cañon. The length of this plug, in a direction transverse to the dam, is two or three times its breadth, and can hardly be conceived to develop the stresses which might be expected in a dam the base of which is as long as its top. The masonry being used is probably heavier and better than that in any of the five dams the profiles of which lie outside it. Also, it is curved to a smaller radius than any of these dams.

The profile of the New Croton Dam is also shown in Fig. 11. This can hardly be called a precedent, for the dam is not completed, and, of course, has not been tested. The design, however, has been before the engineering profession for a number of years and has been approved by many men of the best judgment.

DISCUSSION.

Mr. Frizell.

JOSEPH P. FRIZELL, M. AM. SOC. C. E. (by letter).—A faithful record of the inception, planning and execution of a great engineering work is always instructive to the profession, and, accordingly, this paper by Messrs. Harrison and Woodard on the Lake Cheesman Dam is welcomed. Moreover, the failure of an important work is often more instructive than its successful execution, and in this view, it may be said, without any disparagement to the general scope and purpose of the paper, that its most valuable feature is the account of the failure of the original rock-fill dam. This was a most unfortunate occurrence, not only as a serious pecuniary loss to the promoters of the undertaking, but as discrediting, in their minds, and, no doubt, in the mind of the general public, this mode of construction.

The future prosperity of the arid section of the United States, like that of every other section, depends largely upon agriculture, and the latter depends wholly upon irrigation. The conformation of the country, abounding in deep cañons worn in hard rock, calls imperatively for high dams, and the needs of agriculture call with equal urgency for less expensive methods of building than are ordinarily adopted, or are justifiable in works of municipal water supply. These conditions are fulfilled by the rock-fill dam, and this mode of construction must not be brought into discredit by a failure, the result of an obviously avoidable accident. The record of precipitation on the drainage area appurtenant to this dam has been kept for 30 years or more. At the commencement of the work, in 1898, this record, presumably, was available for the preceding 25 years. This showed that high water occurred in the spring and early summer. In May, 1876, 22 years previously, had occurred a rainfall nearly double that of any other month in the record. Under the arrangements adopted, this might have been regarded as certain to overtop the dam. As the dam was expected to occupy two years or more in construction, the engineer accepted a chance, of at least one in ten, of its destruction while in process of building. It is true, he hoped that such another rainfall as that of May, 1876, would not occur until after the completion of the dam, but, as matters fell out, it did occur. Natural agencies worked without any regard to the engineer's wishes. Many dams of this construction might have been undertaken under chances equally or more adverse, and brought to successful completion. In this case, the fatal event happened, and the dam failed. This occurrence does not invalidate the principle of the rock-fill dam. It only shows the necessity of guarding against the danger to which the loss of this dam is due.

There are two general methods of making such a dam safe while in process of construction: (1) To construct the efflux passage with

Mr. Frizell.

dimensions sufficient to take off the whole flow of the stream in the highest stage, under the head that can be obtained by such a low dam as can be built above the main dam; and (2) construct the rock-fill dam so that it will admit of overflow at any stage of its construction.

In this case, the first method would be inordinately expensive, and the object would be very likely to be defeated by drift-wood brought down by the first great flood.

The writer has long been of the opinion that it would conduce much to the success of the second method, as well as to the general stability and safety of the rock-fill dam, to fill the interstices of the latter with gravel or earth. The work might be executed in layers of 18 to 24 ins. in depth, and, after the completion of a layer of stone, earth might be sluiced in, or deposited, under the action of water, until the latter would stand upon the mass. The construction indicated by Fig. 12 is submitted as a method, though very possibly not the best, of constructing the dam so as to admit of overflow at any stage of its progress. This contemplates a massive crib of squared or flatted timber, notched, fastened together with screw- and drift-bolts, loaded with stone, and decked with thick planking. This is represented in Fig. 12 as 12 ft. deep, 20 ft. up and down stream, and extending across the cañon, which is some 30 ft. wide at the level of the deck. On the up-stream side of this crib the stone, or stone and gravel, filling is commenced. As this proceeds, the rods, *r*, *r*, *r*, are inserted, anchored to the wooden blocks, *a*, *a*, *a*, at the bottom, and adjusted to receive the sleepers, *b*, *b*, *b*, which are embedded in the rock filling, and solidly confined by nuts and washers. To these sleepers, a layer of 3-in. plank is spiked, not necessarily in contact, but leaving intervals of 2, 3 or 4 ins. In carrying forward this work, the aim should be to keep the uppermost sleeper some 6 ft. higher than the general level of the dam; at least, this should be the condition when the water comes to the existing level of the dam. It is obvious that such an event could not occur without warning and time for preparation.

Suppose, in the case under consideration, that the work is up to where the valley is 150 ft. wide. Assume that the maximum flood seeking egress is 3 000 cu. ft. per second; 20 cu. ft. per second per foot of length of the overflow, involving a depth on the crest of somewhat more than 3 ft. The water is then some 9 ft. in depth on the unprotected filling, and moves across it with the very moderate velocity of a little more than 2 ft. per second, a velocity incapable of dislodging the smallest stone. The sketch, Fig. 12, it will be noticed, contemplates the laying up of the stones under the uppermost sleeper in an orderly manner in preparation for an overflow. By turning up the nuts at the heads of the rods, a pressure can be brought upon these stones, rendering them immovable.

Mr. Frizell.

It will be obvious that the danger of overflow diminishes rapidly as the dam rises, but is much greater in the early part of the work than later on, for these reasons: (1) Because, in the latter case, the flood is more largely expended in filling the reservoir; (2) because the discharge through the efflux passage is greater, owing to the increased head.

This mode of protection, applied to the case under consideration, would have involved something like 1 000 000 ft. of lumber, of an inferior grade, and not more than 400 000 lbs. of iron.

The paper is somewhat incomplete as regards the efflux arrangements. Under so great a head, and with so large a volume of water, a more detailed description of this feature is desirable. The inquiring reader might at least desire to understand the necessity for providing three efflux passages at different depths. Does this disposition rest upon any substantial reason, or upon a mere tradition of the engineering profession?

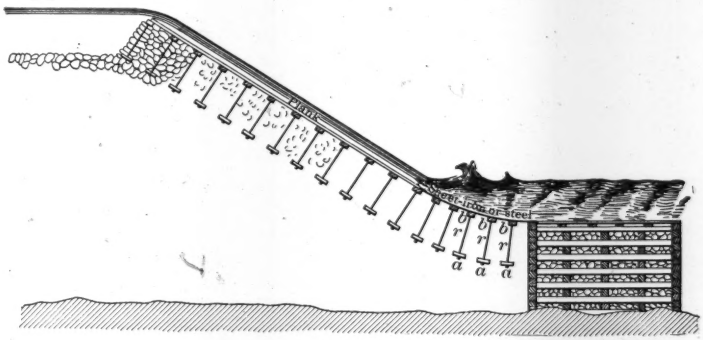


FIG. 12.

Neither the statical principles of the stability of walls nor of arches can be given full effect in the discussion of this structure, though the latter apply much more fully than the former. It is customary to say, in objection to the arched form of dam, that a ring of the arch cannot transmit to the abutment the pressure sustained by it without breaking its connection with the ring below, and that the dam, as a whole, cannot transmit its pressure without breaking its connection with the bottom. This proposition should be more carefully considered.

Regard the dam, with Mr. Woodard, as made up of a series of rings. Consider the action on the upper ring, assumed to be level with the water at the top. It is only in virtue of some slight deformation of the ring that its pressure can be transmitted to the abutments, and this deformation, it will be said, is resisted by the ring below,

Mr. Frizell. through friction and the tensile or shearing strength of the mortar. Remember, however, that no part of the bottom or sides of the cañon is entirely unsusceptible of deformation. Now consider the second ring. This sustains a greater pressure than the one above it, and its tendency to deformation operates to transmit the pressure on the upper ring more fully to its abutment. The pressure on the third ring has the same tendency, and so downward. The deformation of each ring is aided by its connection with the ring above, and resisted by its connection with the ring below, and it takes deformation approximately, but not fully, equal to that due to the water pressure acting directly on it. The direct pressure on the ring, not transmitted to the abutment, is transmitted to the bottom and sides of the cañon, through friction and the tensile and shearing strength of the mortar. It results in a deformation of the material different from any that occurs in the straight dam or the vertical arch: a deformation, namely, in which the upper face of any block tends to move with reference to the lower face.

The computation of the stability of a straight dam of the given cross-section, and in the given situation, without considering any forces other than the weight of masonry and pressure of water, is merely a conventional and academic exercise. Not that such a computation would lead to an unsafe section, but that it ignores forces larger than those of which it takes account. Apply such a computation, for instance, to the part, about 30 ft. wide, that rests on the bottom of the cañon and sustains the entire head of water. This part cannot be overturned without breaking its connection with the remainder of the dam. There is found a pressure of some 45 000 000 lbs. tending to overturn this portion. If the shearing strength of the masonry is estimated at 100 lbs. per square inch (say, for convenience of computation, 1 500 lbs. per square foot), the average thickness of the masonry being something like 100 ft., it is found that there is a force of some $220 \times 110 \times 15\,000 \times 2 = 726\,000\,000$ lbs. tending to resist overturning. Or, if the stability of the entire dam be considered as a body, it cannot be overturned without breaking its connection with the cañon along the entire perimeter of the latter. In fact, it cannot then be overturned without breaking and crumbling into many pieces. In this case, the force tending to push the dam down stream is something like 400 000 000 lbs. The shearing strength resisting this movement is something like 1 260 000 000 lbs. Certainly, if computation shows a margin of safety in disregard of these enormous forces, the dam ought to be safe.

To give an arched form to a dam involves much additional expense and labor, and it would seem that this expense and labor ought to be attended with a diminution of material. Of course, the engineer's aim is to secure safety beyond the possibility of a doubt, but to go beyond this point involves a waste of material. Where dimensions

are adopted which make the structure abundantly safe, regarded as a straight dam, it seems to be unnecessary to give it a curved form. Mr. Frizell.

The furnishing of the cement by the owners, instead of requiring the contractor to furnish it, was a highly commendable feature of this work. No stipulations of a contract are more easy to evade and more difficult to enforce than those which relate to the mixing and the use of mortar. Where the contractor furnishes the cement he is under constant inducement to withhold it from the work. Though he may, ostentatiously and loudly, enjoin upon his men full compliance with the terms of the contract, they readily discern his real wishes, and comply with them. There is no security for the proper use of mortar, unless the use is made to coincide with the contractor's interest.

BURR BASSELL, M. AM. SOC. C. E. (by letter).—The writer has read this paper with pleasure, and, while he does not wish to make a technical discussion of the paper, his interest in the construction of dams in general, and his familiarity with different types and profiles of dams, leads to a few remarks. Mr. Bassell.

In Fig. 11, Mr. Woodard has shown comparative sections of eight dams, including the Lake Cheesman Dam. The writer regrets that the Lake Hemet Dam, California, was not selected instead of the smaller Sweetwater Dam, for two reasons: In the first place, the profile across the cañon at the Hemet Dam site does not differ materially from that of the Cheesman Dam site, after eliminating the lower 30 ft. of the latter, or, in other words, they are practically the same between Elevations + 20 and + 170. Again, the Hemet Dam is about the same height as the Ban Dam, in France, designed to be 165.5 ft. above the lowest foundation, or 152.5 ft. above the creek bed.

The base elevation of the latter corresponds to Elevation + 70 of the Cheesman Dam, at which point they are within 0.51 ft. of the same thickness. The Hemet Dam has a very simple profile or section, having a straight batter of 1 in 10 on the up-stream face and 1 in 2, or 5 in 10, on the down-stream face. Plotting this profile on Fig. 11, and producing the face line downward for deeper foundations, or to correspond to a dam of the same height as the Cheesman Dam, the following points may be noted by way of comparison: The Hemet and Cheesman Dams are of practically the same thickness at Elevations + 70 and + 207 (Cheesman) or Elevations 0 and + 137 (Hemet), being 100 ft. and 18 ft., respectively.

The Cheesman Dam is 176 ft. thick at its foundation base, while the Hemet Dam section would give 148 ft., or 28 ft. less. This means that in the lower 80 ft. of the Cheesman Dam there is a sectional area of 1 120 sq. ft. in excess of the Hemet section. On the other hand, in the upper 150 ft. of the Cheesman Dam, corresponding to the Hemet Dam, the latter has an excess of almost identically the same sectional area. For straight gravity dams, this simple section is certainly safe, and for dams curved in plan it is not much in excess of an ideal section.

Mr. Bassell. In the discussion of curved dams, where arch action may be brought into effect, the bold structure of the Bear Valley Dam, California, is always cited. As a matter of record, the writer will here state that he pointed out the secret of its stability, ten years ago, in the columns of *Engineering News*. This dam was built under hydrostatic pressure, insuring a perfect arch. This method of construction might be used to advantage again, but, certainly, never so bold a section.

The writer appreciates the importance of careful computations to obtain both safe and economical sections for a dam, especially if the dam is high or has a long crest. He also realizes fully the importance of sufficient base width and bed-rock anchorage; however, he is inclined to favor simplicity in design, and would not build too close to theoretical lines in the upper portion of a dam, for fear of seismic disturbances and for other reasons. Furthermore, there seems to be less reason for extending the thickness of that portion of the dam which is below the surface of the ground beyond the limits of the simple section suggested than for decreasing it above.

What the writer wishes to point out, and have fixed in the minds of the younger members, is the simple section here presented which agrees so well with approved profiles for masonry dams, and is represented in actual practice by the Lake Hemet Dam, California. He would recommend, for the lower half of a masonry dam of any height, a thickness of six-tenths of the depth from the top, plus the crown width.

Mr. Maltby. F. B. MALTBY, M. AM. SOC. C. E. (by letter).—The writer has been greatly interested in the description of this work, as he is quite intimately acquainted with the locality. In May and June, 1897, he was employed, under H. M. Chittenden, M. Am. Soc. C. E., Captain, Corps of Engineers, U. S. Army, in making some surveys and examinations of reservoir sites in Colorado and Wyoming for the War Department. He also assisted Captain Chittenden in the study of the development of the various projects and in making plans for the proposed dams. Among others, the reservoir site now known as the Cheesman Reservoir was carefully surveyed, and plans for the dam prepared. The report of these surveys, with maps and plans of the proposed structure, was published as House Document No. 141, 55th Congress, 2d Session; and also in the Annual Report of the Chief of Engineers, U. S. Army, for 1898, p. 2815 *et seq.*

The contour map, Fig. 2, is so similar to the one printed in the foregoing report, almost every indention or projecting point in the contours being identical in each, that the writer wonders if one is not a reduction from the other.

A masonry dam was proposed at identically the same point and of the same height. The curve of capacity, given in the foregoing report, is identical with that given in Fig. 3. In making studies for the proposed dam at this site, the writer made, for Captain Chitten-

den, some ninety-five comparative cross-sections of the high dams of the world. Mr. Maltby.

A rock-fill dam with a steel core, similar to the Otay Dam, in California, was considered, but it was thought that the vast interests of both life and property which would be endangered by the failure of a dam of such magnitude as proposed at this point prohibited the building of any structure which contained an element of danger. For this reason, a masonry dam was proposed. It was to be 200 ft. in height above original low water, or with a total height of about 220 ft. It was proposed to build it on a curve, in plan, having a radius of 300 ft. It is noted that the dam, as described and built, has a curve with a radius of about 400 ft. It was also proposed to make the spillway over a natural saddle of rock, as described. The section proposed was very similar to the one built, having a width of 20 ft. on top and 175 ft. on the bottom, as compared with 18 and 176 ft. as built.

It is a source of gratification to the writer to know that plans for this structure, as given in the above-mentioned report, agree so closely with the plans of the structure as finally designed and built by such eminent men in their profession as were employed as Chief and Consulting Engineers.

E. SHERMAN GOULD, M. AM. SOC. C. E.—This paper, taken in connection with recently published descriptions of similar structures in Australia and California, is of great value at the present time. It seems likely that the high masonry dam of the future will be of the arch type, receiving, sustaining, and transmitting to abutments the stresses acting upon it, rather than opposing them by its mere dead weight, or the moment of its dead weight. In this, the art and science of dam building is but following the universal drift of all branches of construction, in reducing the amount of material used by a more intelligent distribution of it. This is accomplished, generally, by analyzing the stresses acting upon the structure, and framing it of pieces the axes of which coincide with the lines of stress. Also, in the choice of the material used; the growing substitution of light forms of reinforced concrete for massive rubble being a step in this direction. Mr. Gould.

The structure described in the paper is peculiar in that the arch form seems to have been adopted more with a view to fit the work to the ground than for any other motive. Seeing that the dam, if started straight from one end, must be deflected in order to reach proper ground at the other, it was preferred to meet this necessity by a curved line rather than a broken one, in order to secure whatever additional strength the arch form might give it. If it had not been for the peculiar topography of the site, it appears that a straight dam, with what for convenience is termed a "gravity section," would have been adopted, and, indeed, such a section was adopted and

Mr. Gould. counted on to sustain the pressure, the arch being considered only as incidental and merely affording whatever additional strength might reside in it.

Now, the speaker considers that this combination is radically faulty; that the two systems, arch and gravity, do not aid but rather conflict with each other, and that a section which is right for one is wrong for the other. He does not wish to be understood as saying that if an arch dam, with its lighter section, were sprung across a valley, its stability would not be increased by adding to its base a further mass, such as would be required to give it the form of a gravity section; what he does wish to say is, that if such additional material were put into the work, it could be more advantageously placed elsewhere. Furthermore, he would wish it to be understood that in all that he has said, and is about to say, he refers to those cases where the curve is so pronounced as to form a veritable arch, and not a mere camber.

When a dam with a gravity section is built on a curve, its strength to resist crushing or overturning as a gravity dam is impaired, although the deficit is more than compensated by the setting up of the arch action. Therefore, it cannot be considered as possessing all its gravity strength with all its arch strength super-added. The total hydrostatic pressure against it is increased in the ratio that the arc of the curve is greater than its chord, while the resisting area at the toe is reduced, measurably, in the same proportion.

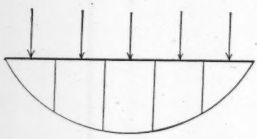


FIG. 13.

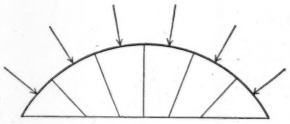


FIG. 14.

These conditions may be seen by reference to Figs. 13 and 14, which show two ideal though quite possible plans, Fig. 13 being that of a straight dam and Fig. 14 that of a curved one, both having the same span. The total pressure upon the curved back is, in this case, six-fifths of that on the straight back. If Fig. 14 is developed, it gives Fig. 15, which shows that the resisting area of the dam is diminished by the gores left vacant by the development.

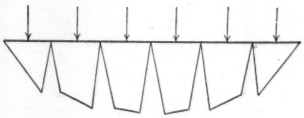


FIG. 15.

But when the arch form is frankly invoked, as in the present case, why should the gravity feature be retained in the problem?

Mr. Gould.

The forces with which the dam contends are: sliding forward on its base; overturning around the outer edge; and, in the case of very high dams, the crushing of the material when the dam is sustaining a full head of water. If the dam is straight, and is cut into imaginary slices, say, 1 ft. thick, all of these phenomena may be produced upon a given slice without disturbing the adjacent ones. In a curved dam this could not take place without setting up violent lateral reactions because the imaginary slices, having their axes in the same lines, as the pressures are no longer parallelogramatic, but wedge-shaped, and cannot be thrust forward or made to rotate without pushing aside or crushing the adjacent ones. It seems to the speaker, therefore, that a gravity section implies a faulty design when applied to an arched dam, because it provides for a class of stresses to which the structure cannot be subjected.

In the speaker's opinion, the calculation for this dam should have been made wholly upon the arch proposition. Mr. Woodard states that this calculation was made, but does not give the method used nor the results, which fact must be taken as a proof that he considered them satisfactory.

The speaker is not aware what method of calculation has generally been used for arched dams, considered simply as such. In some respects it would seem that the calculation must be simpler than for an ordinary arch sustaining a vertically acting load. The weight of the dam itself is not borne by the arch, and the pressure that it does sustain is uniform per unit length of surface, and is directed at all points toward the center of the curve. The line of pressure, therefore, as Mr. Woodard states, must correspond with the curve. What is wanted is the horizontal compressive stress per unit of section. Navier's formula appears to be peculiarly adapted to this calculation, because it is based precisely upon the hypothesis of fluid pressure, and is only applied to other cases by a more or less permissible extension. Navier's formula, as is well known, is as follows:

$$Q = P\,R' \qquad (1)$$

In this formula, referring to Fig. 16, Q = the compressive stress upon any horizontal strip, of length, x, and unit width, the unit in this case being 1 ft., situated at the depth, h, below the surface of the water which is assumed to be level with the top of the dam. P = the horizontal pressure at the depth, h, upon the unit surface (1 sq. ft.), and R' = the radius of the curve of the intrados of the arch. In this case $R' = R - x$; R being the radius of the crest of

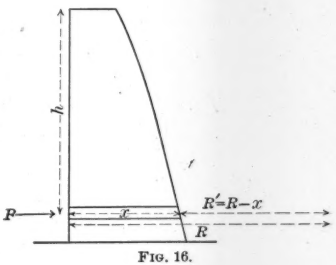


FIG. 16.

Mr. Gould. the dam. R, of course, is constant, while R' varies with the value of x.

The pressure upon the unit of surface, that is, upon 1 sq. ft., due to the head, h, is 62.50 h. The formula then becomes:

$$Q = 62.50\ h\ (R - x) \ldots\ldots\ldots\ldots\ldots\ldots\ (2)$$

To find the proper value for x at any given depth, h, below the top of the dam, a value for Q, based upon the permissible unit stress, must be assumed. The value of Q would then be this stress multiplied by x, since x, being of unit width, is the section which must resist the pressure. Designating this limiting stress by L, we have:

$$L\,x = 62.50\ h\ (R - x);$$

whence

$$x = \frac{62.50\ h\ R}{L + 62.50\ h} \ldots\ldots\ldots\ldots\ldots\ldots\ (3)$$

This is the general expression for the value of x at any given depth, h, when the radius, R, and the limiting stress, L, are given.

It is evident that a high value may be assigned to L, because the pressure, particularly near the base of the dam, is exercised, not upon imaginary isolated strips, as has of necessity been assumed in the formula, but upon strips embedded in a mass of masonry. $L = 30\,000$ lbs. would be a conservative value, under the circumstances. In the Lake Cheesman Dam, $R = 400$ ft. Inserting these values in Equation 3 and reducing, gives:

$$x = \frac{400\ h}{480 + h} \ldots\ldots\ldots\ldots\ldots\ldots\ (4)$$

This is the working formula adapted to the present particular case, and by its aid a series of values of x can be calculated, as in Table 4.

TABLE 4.

Elevation.	h	x	Actual length.
0	220	125.70	166.53
50	170	104.60	119.36
100	120	80.00	72.66
150	70	51.00	37.00
200	20	16.00	18.00

The profile thus determined is shown in Fig. 17, upon which the actual profile of the Lake Cheesman Dam is also shown, approximately, in dotted lines.

Now, whether or not the value of L is rightly chosen, whether the resulting profile, as shown in Fig. 17, is too bold or not bold enough, the fact which the speaker is seeking to establish is that, at least in his opinion, the proper profile for an arched dam is

Mr. Gould.

of the general shape shown in Fig. 17, rather than the one actually used, and it is upon this point, for his own enlightenment, that he would be glad to know the opinions of others.

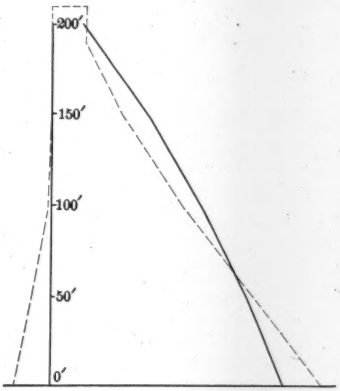


FIG. 17.

Also, he would suggest the following questions as needing further elucidation:

Can the thrust of the water in the arch dam be considered as being transferred totally to the sides of the valley, and, if so, is it necessary to consider the vertical pressure upon the base of the dam as anything more than that due to the weight of the dam itself, when the reservoir is empty? The extreme thinness of some existing, high, arched dams would seem to indicate that it is not.

It would require much more time than is now at command to follow and verify the skilful, but intricate calculations and assumptions developed by Mr. Woodard. The results shown in Fig. 10 are surprising in that they exhibit so slight a difference between the lines of pressure for a full reservoir, with and without the aid of the arch. They seem to indicate that the very pronounced arch form added practically nothing to the strength of the dam.

Against the heavy profile of the Lake Cheesman Dam there must be set off the many very light arch dams already built and standing intact. Apparently, there is discord between theory and practice in regard to structures of this class. If calculation shows that these dams are too frail to sustain the forces acting upon them and they yet persist in standing up, then it would certainly seem that there is something wrong with the theory upon which the calculation is based.

One fact it is well to bear in mind, as explaining the frequent disagreement between fact and theory, in the case of retaining walls in general, and of dams in particular, and that is the enormous difference between the assumed data and the actual conditions, resulting in a great but neglected factor of safety. When the resistance of a gravity dam is calculated, a slice 1 ft. thick is taken at the point where the dam is the highest, and is considered as standing alone, without any lateral support, and depending for its stability wholly upon the moment of its mass.

Then (neglecting besides the adhesion of the mortar), this slice is

Mr. Gould. considered as representing the entire dam. If the dam were of equal height throughout and of indefinite length, and if it fell, if it did fall, in one body, as a door turns upon its hinges, this would be a correct assumption. But no dam was ever overturned in this way, nor could it be. There is no continuous edge about which it could rotate bodily. The valley across which it is thrown is more or less V-shaped, and the whole structure could not be made to rotate about the comparatively small fulcrum which the topography of the site afforded. Every foot of the dam would have a different point about which it would tend to rotate, and no single slice or section could rotate without tearing asunder its connection with the adjacent sections. An enormous proportion of the overturning force would be absorbed in the shearing effort necessary to accomplish this, and of this no account is taken, the result of our calculations, therefore, being tremendously on the safe side. No doubt many anomalous conditions may be thus accounted for.

Something is said in the paper about strains due to changes of temperature. It appears to the speaker that, considering the great thickness of such structures, entirely too much importance is attached to this feature. The effects of changes of temperature cannot penetrate beyond a very few feet, leaving the temperature of the mass practically uniform at all seasons of the year.

Mr. Harrison. E. W. HARRISON, M. AM. SOC. C. E.—The most salient feature which strikes the speaker, in reading this most admirable description of a beautiful piece of engineering work, is the illustration it gives of, probably, the highest type of engineering skill and ability—the art of seizing upon natural features and conditions, and utilizing them to the best and most economic advantage for the purpose sought to be obtained.

Secondly; how often skilful engineers fail to grasp these opportunities, and, in the very conception of a plan, go wrong and produce results far below the possibilities lying open to their brains and hands.

To use an illustration familiar to all engineers who have studied, from an observation car, or as delineated upon a topographical map, a railroad located in a difficult country: how many millions of dollars could have been saved in construction, maintenance and operation, if the locating engineer had only possessed that useful sense of imagination which sees, ahead of the transit, the completed grade winding around the mountain sides and crossing the summits, and makes the configuration of the earth surface assist him in obtaining the location which cannot be bettered.

In the present case there is the condition of very small rainfall, and, what there is, occurring during a short fraction of the year. This entails large storage, and, if possible to obtain, deep reservoirs with little surface for evaporation. To avoid waste, the dam and reservoir

should be as tight as skill can make them, as every gallon saved from leakage and evaporation, prolongs the period in which the available supply will suffice for the growing demand, and increases the growth of the sinking fund which, some day, will be required to extend the supply. Mr. Harrison.

The contour map shows the selected reservoir site to be perfect to fulfill these conditions, and the location chosen for the dam is ideal.

The speaker, in the past ten years, has had occasion to make many studies of the topography in different parts of the country for the purpose of selecting advantageous and economical sites for storage reservoirs and dams, and cannot remember any example in which the combination of all natural features, except rainfall, needed for a water supply, and to secure large storage and a deep reservoir advantageously, were so combined as in the case under discussion.

Yet, after selecting a narrow gorge with solid granite sides and floor, where Nature had provided an ample and excellently located spillway, with quarries of the very best material near by—in fact, a theoretically perfect location for a masonry dam of the most economical proportions—all these advantages were nearly thrown away, and work was actually commenced on a rock-filled dam, faced with a sheet of concrete and steel plates.

Fortunately, for the enterprise, the flood of May, 1900, destroyed the work on this structure, and, presumably from the paper, resulted in bringing into the project new engineering talent, which saw the opportunity presented, and was wise enough to grasp it.

The proper character of the work being adopted, the actual carrying out of the plans presents no remarkable features, and is a matter of detail.

The section for the dam is bold, and, considered as a gravity dam, is only just on the line outside of which the best precedents forbid an engineer to go, but the designer seems to be justified in his confidence, not only by the fact of the unexcelled character of the foundation and abutment rock, but also by having the security of the arch form as a final resort.

Compared with the section adopted for the Boonton Dam, with which the speaker is connected, the section of the highest 110 ft., of the Boonton Dam has an area of 3 836 sq. ft., as against 3 254 sq. ft. in the upper 110 ft. of the Lake Cheesman Dam.

Under any but exceptional conditions of situation and other advantages, as found in this case, the speaker would feel easier with the larger section, though the Boonton Dam, for many reasons—among others the resistance to ice pressure—was treated in a very conservative manner.

From the information given in the paper, the speaker is of the opinion that a saving in cost could have been secured by using the Cyclopean, rubble-concrete masonry which was adopted at Boonton.

Mr. Harrison. The cement estimated as needed, 80 000 bbls. for 103 000 cu. yds. of masonry, or more than 0.77 bbl. per cubic yard, exceeds the proportions actually used at Boonton, which was 0.68 bbl. per cubic yard for more than 200 000 cu. yds. of masonry of all classes..

The weight of a cubic foot of masonry in the Lake Cheesman Dam, is given at 158 lbs., which in itself is excellent, but the average weight in the Boonton Dam—syenitic granite, in concrete—is 166 lbs per cubic foot.

Mr. Horn. FRANK C. HORN, M. AM. SOC. C. E. (by letter).—The sand used in the construction of the dam was washed after passing through a screen made of parallel bars spaced ⅜ in. apart. The grains ranged in size from the largest that would pass through the screen to fine sand. About 34% would be retained on a No. 6 sieve and 66% would pass through it. A number of tests were made to determine the voids, which were found to be about 31.5% of the volume.

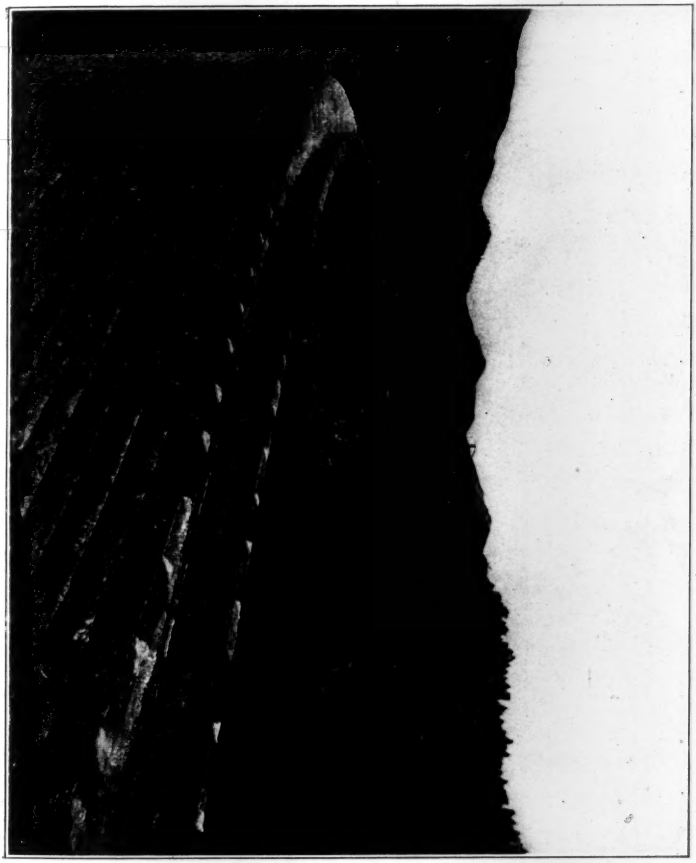
The mortar used in pointing was mixed in the proportions of 2 parts of screened sand to 1 part of cement. The mixing was done in batches of 5 cu. ft., by a batch mixer. The operation of mixing the mortar was watched by an attendant, and a uniform product was always secured. The mortar boxes were each of 10 cu. ft. capacity, and, after being filled, the mortar was distributed as the necessity of the work demanded, and was not allowed to accumulate on the wall. The contents of each mortar box were used within about 30 minutes after having been placed for the masons. Tests were made, from time to time, to determine the tensile strength of the mortar. The results are shown in Table 5.

TABLE 5.—TESTS OF STRENGTH OF MORTAR, LAKE CHEESMAN DAM.

2 to 1 mortar.			2½ to 1 mortar.		
Age of briquette.	Number broken.	Average tensile strength.	Age of briquette.	Number broken.	Average tensile strength.
7 days	75	373 lbs.	7 days	128	320 lbs.
28 "	110	487 "	28 "	185	462 "
6 months.	45	562 "	6 months	105	527 "
.....	.....	.....	1 year	64	581 "

The mortar for making tests was taken from the mortar boxes on the wall. The briquette moulds were filled in the usual manner and placed in the shade under a damp cloth for 24 hours after which they were removed from the moulds and placed in water, where they remained until the date of breaking. They were broken immediately after removal from the water. They were subjected, as nearly as possible, to the same treatment as the mortar used in the masonry.

PLATE VIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIII, No. 981.
HORN ON
LAKE CHEESMAN DAM AND RESERVOIR.



LAKE CHEESMAN DAM.

Mr. Horn.

The stones in the up-stream face of the dam average 30 cu. ft. in volume, and those in the down-stream face about 27 cu. ft. Those used as backing range from 6 to 90 cu. ft. in volume. Each stone was set in a full bed of soft mortar, and worked down to an even and firm bearing, until the mortar on all sides was forced out by the weight of the stone and the action due to working it with the bar. All remaining spaces were filled with soft mortar, into which smaller stones were carefully and firmly placed, care being taken to bed in mortar each stone, large and small, and not allow contact one with another. The bonding of the stones in the wall was given particular attention. No leveling of the wall at any point was permitted, except for setting the up-stream face stones, but a thorough bonding, horizontally and vertically, was secured. Fig. 1, Plate V, and Fig. 1, Plate VII, well illustrate the character of the work.

The percentages of stone and mortar entering into the completed masonry are about 72 and 28, respectively.

The masonry, up to this time, is water-tight. Water has stood at Elevation 97.0 for a year, and no leaks, or even sweating, are discernible on the down-stream face of the dam.

Plate VIII is a view of the top of the dam, taken after the completion of the parapet walls, but before finishing the concrete roadway.

Mr. Gowen.

CHARLES S. GOWEN, M. AM. SOC. C. E. (by letter).—This is a very interesting and valuable paper, particularly when it is considered that this dam is designed and has been built to withstand a clear static head of more than 200 ft., and that, from the account of the authors, there was no sign of percolation or sweating under its first test of more than 100 ft. of static head. It seems to the writer that such a result is remarkable, and it is evident that, in the material, methods of construction, and workmanship, the very best of judgment and skill were used. It would be of great interest to know the result due to the increased head which, no doubt, has been used against the dam during the past winter.

It is evident that the section of the dam described in the paper, namely, that below Elevation 100, was not affected by temperature, and that there were no shrinkage or temperature cracks of account, in the masonry, and, if the writer is not misinformed, he understands that since the completion of the structure to its elevation of 217 ft. no shrinkage or temperature cracks have developed in the masonry, beyond two very slight ones which show at either end of the structure, leaving a clear stretch or length of 700 ft. of the thinnest section of the dam in which no cracks are found.

To judge from the description and illustrations, it would seem that the masonry of the dam was carried up uniformly, practically without steps or racks, excepting such as are described as having been used to

Mr. Gowen. break bond in the masonry horizontally. In view of the presence of shrinkage or temperature cracks in the narrow sections of high masonry structures, ordinarily, and of the annoyance occasioned by them, it may be well to consider the reasons why their absence has been so marked in this structure.

At the New Croton Dam, the masonry of which, it may be said, can be divided into three parts, the first comprising the main dam, proper, which is built on a straight line; the second comprising the curved portion of the overfall, which curve is nearly a quadrant, and the third comprising the remaining portion of the overfall, which is built in tangents with two or three angles; the cracks which have developed are particularly noticeable in certain portions of the main dam and the straight portion of the overfall. These cracks, of greater or less size and extent, owing to certain conditions, are confined to the upper or thinner portions of the sections in question, but, in the curved portion of the overfall, which has been subjected to the same conditions as have obtained in the straight portions of the dam, no cracks have been found.

While temperature or shrinkage cracks are likely to be concentrated at particular points in case the masonry is not carried up at uniform elevations, it is unquestionable that, with uniform areas of section, they would occur at intervals more or less regularly in any thin masonry structure compactly built on a straight line. This, the writer thinks, is shown clearly enough in the work at the New Croton Dam, at certain points where the methods adopted resulted in a comparative uniformity of lines of elevation and sections of masonry as they were built. At other points it is evident, from the size and depths of the cracks, that the variations of sections, and longitudinal racking had a great deal to do with the concentration of the cracks. In the masonry of the Lake Cheesman Dam, however, it seems to be clear that its curved plan and the uniformity of section which was preserved throughout its construction had evidently to do with the remarkable absence of temperature cracks which has been noted, and the writer thinks it may be said that these results constitute a pretty good argument in favor of building high masonry dams on curves, if circumstances are such as to allow it without an undue increase in expense.

Another point of interest in connection with this dam is the provision made for waste by means of the overfall, the length of which is stated to be 300 ft. Apparently, this overflow will pass about 10 000 cu. ft. per second, a flow approximately five times as great as the largest observed flow of the river in thirty-two years. As a matter of comparison, the overflow of the New Croton Dam is 1 000 ft. long, and the probable maximum depth of flow over it may be assumed at 6 ft. Such a depth would give a flow of approximately 45 000 cu. ft. per

second, or about three times the observed maximum flow of the Croton River for the past sixty years. If it be assumed that the possible run-off from the Croton Basin above the New Croton Dam is equivalent to a rainfall of 6 ins. in 24 hours, or at the rate of $\frac{1}{4}$ in. per hour over the whole water-shed of 360 sq. miles, the run-off would be 55 000 cu. ft. per second, which is equivalent to a depth of about 7 ft. on the overflow. Mr. Gowen.

While an overflow capacity of five times the maximum flow of the river, as observed through a somewhat extended time, may be considered a conservative provision, and while, of course, there can be no comparison properly between the possible run-offs from the Lake Cheesman water-shed and the Croton water-shed, owing to the great difference in areas and rainfalls, the comparison between the provision made in either case, based upon observed maximum flows, is interesting, and the writer, therefore, calls attention to it. As stated, no comparison can be made as regards possible maximum flow, but, if it be assumed that this rate of run-off is in inverse proportion to the water-shed areas, then, on the basis of the 6 ins. in 24 hours, assumed in case of the Croton, 6 divided by 5 equals 1.2 ins. From the Lake Cheesman water-shed, the run-off, based upon these figures, would be approximately 63 000 cu. ft. per second.

Continuing the reductions further, and in proportion to the respective mean yearly rainfalls of the two basins, 48 ins. for the Croton and 14 ins. for the Platte, then, approximately, the possible maximum is 18 000 cu. ft. per second, a result which, if considered seriously, is seemingly out of proportion to the length of overflow provided.

The records of the United States Geological Survey show flood flows of certain streams in Kansas and Colorado which, apparently, approximate closely, if they do not pass, the limit of 10 000 cu. ft. per second for the 1 800 sq. miles of the Lake Cheesman water-shed. Accepting such records as reasonably reliable, it would seem that greater flows might be expected than this overflow is calculated to provide for.

An acquaintance with the characteristics of this particular water-shed is certainly essential in order to justify any criticism of the feature in question, and the writer, therefore, is not disposed to undertake it; but if the information on which the dimensions of the overflow were based can be supplied, it would prove a very acceptable and valuable addition to this paper.

The plans made for regulating the outflow from the reservoir were formed, it is evident, in connection with the rock-fill dam design and were partly completed before the new design for a masonry dam had been adopted. The tunnels, it is assumed, are unlined, excepting at the points where the valves are installed, and the question arises as to the need, in time, of additional tunnel lining, and it is certainly a

Mr. Gowen. matter of great interest to know how the valves in question are working under the pressures that must obtain. The addition of the Tainter guard-valve was certainly very essential, and the precautions taken to provide the manway tunnels to give access to the working parts of the discharge valves at all times were certainly prudent, to say the least. It is improbable, of course, that all three, or that two out of three, valves will be out of order at any one time, and the contingency is remote that the control of the flow might be lost in consequence, or that in such case the depths of the water above the dam would be beyond the limit of a diver's work, but it would seem to the writer that some provision should have been made, if such has not been done, by which the upper entrances at the 60-ft. and 110-ft. levels can be closed upon emergency and clear access to the valves thus afforded.

Mr. Wegmann. EDWARD WEGMANN, M. AM. SOC. C. E.—The paper shows that the Cheesman Dam has been well designed and built. It will be the highest structure of its kind until the New Croton Dam shall have been completed. The profile of the dam was determined on the principle that the dam was to resist the water pressure by its weight alone, and the curving of the plan of the wall appears to have been done principally to obtain an economical location, as is shown clearly on Plate IV. That this arching adds materially to the strength of the structure to resist overturning, will be admitted readily by most engineers.

Two dams have been constructed, which owe their stability solely to the fact that they are curved in plan, so as to act as horizontal arches, *viz.*, the Zola Dam, in France, and the Bear Valley Dam, in California. The first of these dams was built in 1843 for supplying the City of Aix with water. It has a maximum height of about 120 ft. and a length on top of only 205 ft. At the top and bottom the dam is, respectively, 19 and 42 ft. wide. The dam is curved in plan, the radius at the crown being 158 ft. When the reservoir is full, the line of pressure at the base falls 11.48 ft. outside of the structure, which, therefore, would be overturned, if it were not curved in plan.

The Bear Valley Dam, built in 1884, has a maximum height of 64 ft. It is 3.17 ft. wide at the top and 20 ft. wide at the base. The plan of the dam is curved, with a radius of 335 ft. When the reservoir is full, the line of pressure at the base falls about 15 ft. outside of the dam.

It is a simple matter to determine the profile of a dam which is to resist either solely by gravity or only as an arch. However, when both actions are assumed as occurring at the same time, and an endeavor is made to calculate the relative resistance offered by each, the problem becomes quite complicated. The French engineers, Delocre and Pelletreau, gave solutions* of this problem, more or less

* *Annales des Ponts et Chaussées* for 1866, 1876 and 1877.

satisfactory. To these must now be added the solution by Mr. Woodard, given in the latter part of the paper. Mr. Wegmann.

In applying mathematics to this problem, we are obliged to assume, in our present state of knowledge, that the masonry of the dam is perfectly elastic, which causes the strains to be according to a "uniformly varying stress." That this assumption, when applied to a masonry dam 150 ft. thick at the base, may be far from the truth is recognized fully by Mr. Woodard. While complimenting him on the mathematical skill he has shown in dealing with this intricate problem, the speaker questions whether any reliance can be placed on the results obtained by formulas based on such uncertain assumptions. Professor Rankine, in his "Report on the Design and Construction of Masonry Dams,"* reached the conclusion that there was such uncertainty in trying to calculate the resistance offered by a curved dam, due to its weight and the arch action at the same time, that he recommended that a dam be always made sufficiently strong to resist the water pressure by gravity alone; and that, in case of a narrow valley, the plan be curved so as to give some additional, though unknown, strength to the structure.

This is exactly what was done in the case of the Cheesman Dam, and Mr. Woodard's formulas appear to have been an after-consideration to determine just how much resistance was offered by the weight of the dam and how much by the arching, both being assumed to act at the same time. If these formulas, however, are to have their full practical value, they must be considered sufficiently reliable to enable us actually to reduce the profile of a "gravity dam," when curved in plan. Probably few, if any, engineers would be willing to apply Mr. Woodard's formulas to that extent.

Mr. Woodard states that the usually accepted method of determining the profile of a dam resisting the water pressure by gravity alone is simple and, with variations in details, may be found in textbooks and works on masonry. Twenty years ago, when the calculations for the New Croton Dam were made, this was by no means the case. The only methods that could be found were those of the French engineers, Sazilly, Delocre and Pelletreau, involving equations of the fourth and sixth degree, and that of Professor Rankine, for a profile bounded by logarithmic curves. All these engineers designed "profiles of equal resistance," *i. e.*, profiles in which the maximum pressures at the faces of the dam were kept at a uniform limit. From this resulted profiles with curved faces.

In designing the New Croton Dam, the principle followed was to limit the lines of pressure, reservoir full or empty, to the center third of the profile, allowing the pressures at the faces to increase gradually until they reached the limit of safety, which had been assumed.

* See *The Engineer*, January 5th, 1872.

Mr. Wegmann. Below this point the slopes of the faces were increased so as to keep the maximum pressures at the limit of safety. By simple equations of the second degree, the thickness of the dam was calculated at intervals of 10 ft. This gave a profile with polygonal faces, having many changes of batter, for which, in the final design, curves, or a few straight lines, were substituted.

This method, which was first published in 1887, in the Report on the Quaker Bridge Dam, by the late A. Fteley, Past-President, Am. Soc. C. E., Consulting Engineer of the Aqueduct Commissioners of New York, and in the following year, by the speaker, in a book on dams,* appears to have been adopted in the design of most high American dams since that time. From some of the details given in the paper and from the general look of the profile, the speaker concludes that the Cheesman Dam was designed by the method just mentioned.

Mr. Harrison has described the high rock-fill dam which was begun at the site of the Cheesman Dam and was destroyed by a flood. Considering the large cross-section which had to be given to the rock-fill, it is evident that such a structure could only be justified when the cost of cement delivered on the work was very high, and it would be interesting to know what the difference in cost would have been between the rock-fill and the masonry dam.

The details of the outlet tunnels, valves, etc., show skilful engineering.

The authors are to be thanked for their full description of this important high dam, and for giving the profession Mr. Woodard's method of treating a curved dam simultaneously as a horizontal arch and a "gravity section."

Mr. Smith. J. Waldo Smith, M. Am. Soc. C. E.—Mr. Harrison has discussed this paper so thoroughly, from the speaker's standpoint, that little more can be said. It would be interesting, however, to hear an answer to Mr. Kuichling's question regarding the temperature strains or the cracks from internal stresses in the dam. This is a subject which is of great interest to engineers at the present time, as several are studying what is happening in dams which have been built during the last eight or ten years in reference to various cracks which have developed from one cause or another.

Measures have been taken to ascertain the temperatures in the Boonton Dam by introducing a number of thermophones, from which the temperatures are to be read during a long period. These were put in during last season, but at the present time sufficient data have not been obtained from which to draw conclusions. It can only be said in a general way that the temperature in the interior of the dam is still falling, and has been falling steadily from the start. The second

* "The Design and Construction of Dams," by Edward Wegmann, M. Am. Soc. C. E.

point shown is that no matter what the temperature of the air at the time the masonry is laid, the ultimate maximum temperature of the masonry is approximately the same. The thermophones near the outside of the wall indicate a steady fall through the cold weather, and, as soon as the temperature changes in the spring, they indicate a rise. The daily changes in temperature seem to have considerable influence on the average temperature of the dam, and, although the average monthly temperature is falling steadily, the average daily temperature shows a rise and fall following somewhat closely the changes in the air. Mr. Smith.

E. KUICHLING, M. AM. SOC. C. E.—This is an extremely interesting and valuable paper, as it presents clearly the essential details of the design and construction of a great masonry dam. In this respect, the author's description leaves little to be desired; but, inasmuch as the hydrology of the watershed under consideration is peculiar, and differs greatly from that of drainage basins of similar magnitude in other parts of the United States, it has appeared to the speaker that somewhat more space might have been given advantageously to the rainfall and run-off of the South Platte River Basin. Mr. Kuichling.

The author states that the maximum flood flow of the river at the site of the dam is only 1 945 cu. ft. per second, from an area of 1 796 sq. miles. This is at the rate of but 1.08 cu. ft. per second per square mile, which is remarkably low when the rainfall statistics for Denver are considered. From the latter are found the following monthly maxima, in inches, during 32 years (1872–1903):

January.....	2.35	May	8.57	September....	3.70
February....	1.22	June.....	4.96	October..	3.92
March.....	3.10	July	4.28	November....	1.93
April.....	8.24	August....	2.84	December	2.32

and from the much greater altitude of the water-shed, the average elevation of which is about 8 000 ft. above the sea, it may be presumed that the precipitation is somewhat more than at Denver.

It is also found, from the hydrographical records of the United States Geological Survey in this region, that the river freshets usually occur in May and June; and, as the run-off in these months may readily amount to 75% of the rainfall, it follows that from the precipitation of 8.57 ins. in May, 1876, and 4.96 ins. in June, 1882, run-offs of, respectively, 6.43 and 3.72 ins. in depth on the water-shed might have occurred. Now, as a depth of 1.0 in. run-off in 30 days corresponds to an average discharge of about 0.9 cu. ft. per second per square mile, and as the maximum rate of discharge during the month may easily be twice the average, it also follows that, in the two months mentioned, freshets of about 12 and 7 cu. ft. per second per square mile of water-shed might have been expected.

While these figures of run-off are merely speculative, they are much

Mr. Kuichling. less than has been observed from similar monthly rainfalls in the streams which drain the mountainous regions of the Eastern States, and hence a more extended description of the meteorology and physical characteristics of the drainage basin of the South Platte River will add much to the value of the paper. It may also be noted that a run-off equal to a depth of 0.84 in. on the water-shed will fill the reservoir, and that such a quantity may be expected from the above-mentioned maximum precipitations for November, December and January.

Another matter of interest is the author's assumption that during the winter months the reservoir will never be full, but will always be at a low stage, so that any thrust on the dam due to the expansion of the ice on the surface of the water may be neglected. A further reason for not considering ice thrust is the circumstance that a rocky promontory a few hundred feet above the dam will serve as a protection against the push of the ice, even though the reservoir may be filled to near the spillway level.

From the meteorological data, however, it seems to the speaker very possible that the reservoir may become full during freezing weather, and may remain so for a few weeks if the outlet valves are not regulated carefully or if they become disabled. In this event, ice will form at a high level, and, by its expansion when warmed by the heat of the sun during the daytime, a considerable thrust will be exerted against the dam.

As to the magnitude of the thrust which is actually exerted by the ice sheet on a reservoir, few data seem to be available. Some writers have considered that it is limited only by the crushing strength of the ice, which is from 200 to 1 000 lbs. per square inch, or from 14.4 to 72.0 tons per square foot, depending on its quality and temperature; while others have taken the elasticity of the ice into account, assuming a modulus, E, varying from about 180 000 to 360 000 lbs. per square inch, and a rate of expansion varying from about $\frac{1}{1500}$ to $\frac{1}{4000}$, in passing from 0° to 32° Fahrenheit.

Applying these latter figures to the case of a sheet of ice, 500 ft. wide between the dam and the rocky promontory mentioned, it will be found that the elongation will range from $\frac{1}{3}$ to $\frac{1}{8}$ ft., or from 4.0 to 1.5 ins.; and if $E = 180\,000$, the compressive stress in the ice will be from 120 to 45 lbs. per square inch, while, if $E = 360\,000$, the stress will be from 240 to 90 lbs. per square inch. On this basis of computation, the probable stress ranges from 45 to 240 lbs. per square inch, or from 3.24 to 17.28 tons per square foot; and if the ice is assumed to be 1 ft. thick, the thrust against the dam, accordingly, will be from 3 to 17 tons per linear foot.

It may also be remarked that in the consideration of this question the Board of Experts on the projected Quaker Bridge Dam, in the valley of the Croton River, New York, recommended in 1888 that

provision should be made for an ice pressure of 21.5 tons per linear foot at the spillway level; and that, in designing the large dam which is soon to be built across the Scioto River near Columbus, Ohio, Samuel M. Gray, M. Am. Soc. C. E., adopted an average ice thrust of 11.33 tons per linear foot. Mr. Kuichling.

In view of the widely varying data relating to the crushing strength, elasticity and rate of expansion of ice, the speaker endeavored, a number of years ago, to deduce an approximate measure of the thrust of an extensive field of ice by computing the shearing resistance of the masonry piers of several highway bridges which were a very short distance above certain river dams. The water above these dams usually had only a slight mean velocity in winter, but the discharge was then generally sufficient to produce a moderate depth of overflow, thus allowing the ice below the bridge to expand or contract freely, while the long field above the bridge was held by the piers and banks.

In all these cases the piers had remained undisturbed for many years, although the ice was seldom less than 1 ft. thick in midwinter. The weakest piers of the series were 4.5 ft. thick, 18 ft. long, 11 ft. high above the usual level of the ice, and 68 ft. apart, carrying a light iron structure the weight of which did not exceed 550 lbs. per linear foot; hence the total weight resting on the course of masonry immediately below the ice was about 185 000 lbs. The mortar was not strong, and had been partly washed out of the joints, so that a high value for its shearing strength could not be taken. Allowing 5 lbs. per square inch for such shear and 0.6 for the coefficient of friction, the total resistance of the pier to sliding becomes about 169 000 lbs., or 37 600 lbs. per foot of its thickness; hence, in this case, the ice thrust was certainly less than 18.8 tons per linear foot; and, if the total resistance be computed with 0.7 as coefficient of friction including shear, the thrust will be less than 14.4 tons per linear foot.

All these computations for the ice thrust on dams with vertical backs are based on data so crude that the appearance of a better array of observations becomes very desirable; and, in view of the importance of the subject, it is earnestly hoped that those who have had the opportunity to measure this force of Nature will contribute to our stock of knowledge concerning it.

Another matter of interest is the behavior of the masonry of the Cheesman Dam with regard to changes of temperature. The speaker was informed recently that during the past winter several vertical cracks made their appearance in the upper portion of the structure. If this be true, it follows that the method of computing a curved dam by considering it as an elastic and continuous arch must be modified.

R. Shirreffs, M. Am. Soc. C. E. (by letter).—The writer has been greatly interested in the second part of this paper, relating to the Mr. Shirreffs.

Mr. Shirreffs.

stresses developed in a curved dam, not only because he has at present in charge a project where a curved dam is as emphatically "indicated" as it was at the Lake Cheesman site, but also because it fell to his lot a few years ago to direct, under F. P. Stearns, M. Am. Soc. C. E., a study of the merits and demerits of the curved *vs.* the straight form for the Wachusett Dam, then proposed, and now being built by the Metropolitan Water Board, of Massachusetts.

The studies were made in a similar way to those now presented by Mr. Woodard, but with fewer assumptions not borne out by the facts. Unfortunately, the writer's memorandum of these calculations was destroyed by fire some time ago, and what follows has been worked out entirely anew and, it is believed, with some improvement on the methods then followed.

Upon examining the methods followed in the paper, it will be found that several assumptions or approximations are made which do not conform strictly to the actual conditions. The writer believes that some of these can be eliminated without increasing very much the labor of calculation, and with a marked gain to the advantages shown by the curved dam. In the case of the Lake Cheesman project it appears that the only purpose of the analysis was to justify the use of a "gravity" profile when built as a curved dam. The errors occasioned by the assumptions made, as these were nearly all adverse to the vertical beams, were on the side of safety, but they are misleading when applied to the general question of the curved *vs.* the straight dam. And here it may be proper to emphasize the point, to which attention has already been called in the discussion of this paper, that the straight dam is only a special case of the curved dam, and that it is not possible, simply by building a dam straight, to avoid the stresses caused by the combination of vertical with horizontal beams. The real point is whether it is better to build the dam straight, with the certainty that there will be tensile stresses in the horizontal beams, or to avoid all tensile stresses by building the dam on a curve when the site will permit the use of a short enough radius.

The approximations referred to are these:

1.—In ascertaining the deflections of the vertical beams, an average value of the moment of inertia was used for each different vertical section;

2.—The deflections of the arch rings were computed as if due to the constant radius of the extrados of the arches; and, most important;

3.—The arch deflections were computed (*a*) as if there was a hinge point at the abutments, and (*b*) as if, even then, there were no moment stresses in the arch.

As an illustration of the errors resulting from this last assumption, and also as an illustration of the advantage of reducing the radius

Mr. Shirreffs.

when it can be done, the results of the computation of two curved dams of triangular section, No. 1, having a radius of 400 ft. and a face batter of 0.5477 ($b^2 = 0.30$); and No. 2, having a radius of 260 ft. and a face batter of 0.40, are given in Table 6, the first set of figures resulting from the arch deflections obtained by Mr. Woodard's method and the second set resulting from deflections obtained by treating the arches as curved beams. In both sets the approximations noted above under "1" and "2" have been corrected. Both profiles are supposed to be built in a triangular valley similar to that sketched in Figs. 18 and 19, that is, with a maximum depth of water of 180 ft. and a chord width of 500 ft., at a point 30 ft. below the water line, and with the contours opening outward, up stream, so as to be found in the same relative position on the arcs of the curved dams as on the straight line across the valley. The whole height has been divided into five arch slices of 30 ft. each, as in the example in the paper.

TABLE 6.

Joint.		1	2	3	4	5	6
	Q	63 280	112 500	168 750	225 000	281 250	.....
Profile No. 1.....	P	43 600	97 230	156 060	206 780	278 550	.....
		37 650	85 980	147 280	211 070	275 680	.....
	$\frac{u+v}{l}$	.....	0.602	0.679	0.724	0.754	0.782
		.....	0.561	0.628	0.672	0.706	0.736
Profile No. 2.....	P	15 320	59 620	122 360	197 750	271 700	.....
		11 520	41 580	86 100	172 680	261 840	.....
	$\frac{u+v}{l}$	.....	0.469	0.644	0.762	0.867	0.960
		.....	0.434	0.554	0.634	0.733	0.836

In Table 6, Q is the total water pressure, in pounds, considered as concentrated at the various joints; P is the part of this load carried by the vertical beam at the center of the dam, and $\frac{u+v}{l}$ the ratio of the distance of the resultant pressure from the up-stream edge of the joint to the length of the joint. (See also the arch deflections in Table 9.)

It will be seen that the use of the true arch deflections has a considerably greater effect in No. 2 than in No. 1 in reducing the eccentricity of the resultant pressure beyond the "middle third" at the lower joints. It is entirely probable that the necessary modification of either profile to give it a reasonable top thickness will fully correct this eccentricity in either. At this point attention may be called to the superior ability of a curved dam to resist shocks, as of ice or drift, for which a top thickness is necessary; especially in respect to the

Mr. Shirrefs. most dangerous of these effects in a cold climate—the thrust of a field of ice expanding on a bright, sunny day succeeding an extremely cold night. It is probable that this superior ability will justify the use of no more masonry in the top of a curved dam than would be used in a straight dam of the same height, or that, in other words, the top thickness having been determined for a straight dam, the curved dam may be given a lesser thickness in the direct proportion of the chord to the arc.

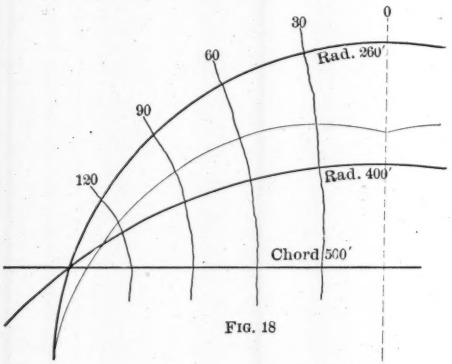


FIG. 18

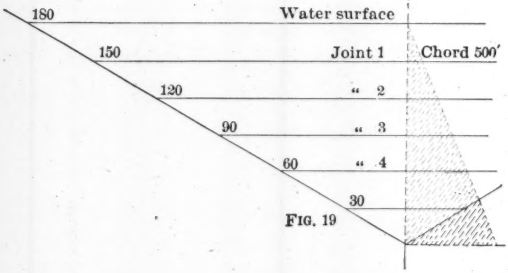


FIG. 19

The volume of Profile No. 1, if built in the assumed valley, will be 67 200, and that of No. 2, 58 490, cu. yd., a difference of 8 310 cu. yd. in favor of the latter, or about 12 per cent. A gravity profile would have a face batter, $b = \sqrt{\frac{1}{\delta}}$; δ being the ratio of the weight of masonry to that of water. Assuming this, with Mr. Woodard, and as has been done in the above computations, to be 2.5, the batter of the gravity profile would be 0.63246. A straight dam across the assumed valley with this section would contain 63 250 cu. yd., while the same

section built on radii of 400 and 260 ft. in the same valley would contain, respectively, 77 120 and 91 880 cu. yd. Mr. Shirreffs.

The economy of the curved dam, however, must obviously be determined specially for each situation. At the Lake Cheesman site, for instance, a straight dam was evidently out of the question. It appears from Mr. Maltby's discussion that a radius of 300 ft. was at one time proposed for this dam, and from the contour plan (Fig. 1, Plate IV), it would seem that a radius as short as 260 ft. would have increased the top length only about 60 ft. and the lengths below Elevation 150 (water surface at Elevation 214) only about 15 ft. as an average. The economy of the curved dam on 400 ft. radius, therefore, would have been about 13%, as compared with a "gravity" section on the same radius; while the curved dam on 260 ft. radius would have saved, say, 36% over the gravity dam on the same radius and about 27% over the curved dam on the longer radius.

In the foregoing computations the dam with triangular profile was chosen because all the mathematics are simpler for this form. It is also the theoretical form for a gravity dam as well as for an arched dam, if, in the latter case, it be considered that the arch elements carry all the load, and if it be further assumed (which is not very far from the truth) that the average normal stress at all points is $S = \frac{q\,R}{A}$. In making the first studies for any new situation it will probably save time to use the triangular profile for the first approximations, afterward making the necessary modifications to give the top a reasonable width. The caution may here be noted that instead of adding to the triangular form a parallel top there should be added only a moderate top width, with a change of the batter for a considerable distance. If a considerable top width be added to the triangular profile the result will be negative loads on the vertical beams toward the top, giving tension at the upper joints at the downstream face.

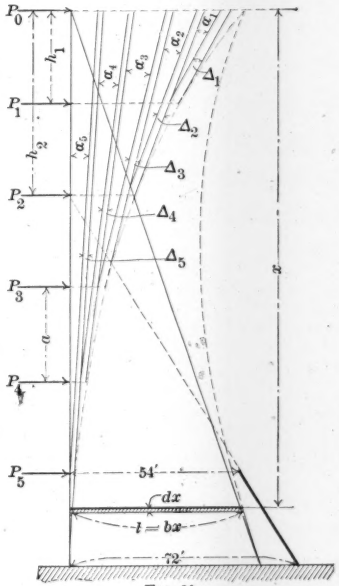


FIG. 20

Passing now to a detailed description of the modifications proposed in the methods used

Mr. Shirreffs. by Mr. Woodard, these will be taken up in the order in which they are noted above.

First.—The following method is proposed to obtain the deflections, under the action of a series of concentrated loads, of a beam having a varying moment of inertia, but a constant thickness = 1.0. It is necessary that the beam be limited by straight lines between the load points. Using the notation shown in Fig. 20, and, in addition, the following:

b = ratio of increase of joint length;
I = moment of inertia of joint;
e = distance from neutral axis to extremity of joint;
M = moment of all loads above joint considered;
σ = stress at extremity of joint;
λ = change of length of element, $d\,x$, of beam, due to σ;
E = modulus of elasticity of material (in compression);
ρ = radius of curvature of deflected beam;
α = angle of deflected beam at any load point, having reference only to the portion of the beam between that load and the one next below;
Δ = deflection of each individual portion of beam, produced by all loads above that portion;
D_n = the total deflection of beam at any load point:

We have, generally:

$$\sigma = \frac{M\,e}{I}\,;\ \lambda = \frac{\sigma\,d\,x}{E}\,;\ \rho = \frac{E\,I}{M}\,,\ \text{and}\ d\,\alpha = \frac{d\,x}{\rho}.$$

But (referring for the present only to the beam of triangular profile),

$$e = \frac{l}{2} = \frac{b\,x}{2}\,;\ I = \frac{l^3}{12} = \frac{b^3\,x^3}{12}\,,\ \text{and}\ d\,\Delta : \lambda = (x - h) : e;$$

therefore,

$$d\,\Delta = 12\,M\,(x - h)\,\frac{d\,x}{b^3\,E\,x^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$\text{and}\ d\,\alpha\ (\text{or also}\ d\tan.\ \alpha) = \frac{12\,M\,d\,x}{b^3\,E\,x^3} \quad \ldots\ldots\ldots\ldots\ldots (2)$$

But we can write $h = n\,a$, and therefore,

$$M_n = P_1\,(x - a) + P_2\,(x - 2\,a) + \ldots\ldots + P_n\,(x - n\,a).$$

The reduction and integration of the resulting equations being quite simple, these pages will not be cumbered with anything except the final results. After obtaining the values of Δ_n and tan. α_n, these must be cumulated as follows for D_n:

$D_5 = \Delta_5$;
$D_4 = \Delta_5 + \Delta_4 + a$ tan. α_5;
$D_3 = \Delta_5 + \Delta_4 + \Delta_3 + a\,(2$ tan. $\alpha_5 +$ tan. $\alpha_4)$;

and so on, as will appear from Fig. 20.

Before assigning values to either a or b, we have :

$b^3 E \Delta_5 = 12 \sum_5^1 P (L n 6 a - L n 5 a) - \frac{1}{30} (61 P_1 + 62 P_2 + 63 P_3$ Mr. Shirreffs.
$+ 64 P_4 + 65 P_5)$;

$b^3 E \Delta_4 = 12 \sum_4^1 P (L n 5 a - L n 4 a) - \frac{6}{100} (41 P_1 + 42 P_2 + 43 P_3$
$+ 44 P_4)$;

$b^3 E \Delta_3 = 12 \sum_3^1 P (L n 4 a - L n 3 a) - \frac{1}{8} (25 P_1 + 26 P_2 + 27 P_3)$;

$b^3 E \Delta_2 = 12 \sum_2^1 P (L n 3 a - L n 2 a) - \frac{1}{3} (13 P_1 + 14 P_2)$;

$b^3 E \Delta_1 = 12 P_1 (L n 2 a - L n a) - 7.5 P_2$; ($L n$ (a) = Naperian Log. (a));

$a b^3 E \tan. \alpha_5 = \frac{1}{150} (49 P_1 + 38 P_2 + 27 P_3 + 16 P_4 + 5 P_5)$;

$a b^3 E \tan. \alpha_4 = \frac{3}{200} (31 P_1 + 22 P_2 + 13 P_3 + 4 P_4)$;

$a b^3 E \tan. \alpha_3 = \frac{1}{24} (17 P_1 + 10 P_2 + 3 P_3)$;

$a b^3 E \tan. \alpha_2 = \frac{1}{6} (7 P_1 + 2 P_2)$;

$a b^3 E \tan. \alpha_1 = 1.5 P_1$.

Assigning to a the value, 30, as in the above and the Cheesman Dam examples, we have after cumulation:

$b^3 E D_1 = 7.3344 P_1 - 3.8501 P_2 - 1.8178 P_3 - 0.6990 P_4 - 0.1545 P_5$;
$b^3 E D_2 = 3.8501 P_1 - 2.5168 P_2 - 1.3178 P_3 - 0.5323 P_4 - 0.1211 P_5$;
$b^3 E D_3 = 1.8178 P_1 - 1.3178 P_2 - 0.8178 P_3 - 0.3656 P_4 - 0.0878 P_5$;
$b^3 E D_4 = 0.6990 P_1 - 0.5323 P_2 - 0.3656 P_3 - 0.1990 P_4 - 0.0545 P_5$;
$b^3 E D_5 = 0.1545 P_1 - 0.1211 P_2 - 0.0878 P_3 - 0.0545 P_4 - 0.0211 P_5$.

Before the arch equations can be combined with the beam equations it will be necessary, of course, to assign a value to b. In applying the foregoing method to a beam of irregular profile, but which can be bounded by straight lines between the load points, it is only necessary to remember that for each beam section there will now be a different origin for x, and a different value for b. For example, if the bottom section of the beam have a joint length of 54 on the top and 72 on the bottom, with a still $= 30$, the origin of x will be at the second load point (see Fig. 20), and the value of b will be 0.60. There should now be written, $x - h = x - 90$, and

$M_5 = P_1 (x + 30) + P_2 x + P_3 (x - 30) + P_4 (x - 60) + P_5 (x - 90)$,

and after the substitution of these values in the general equation (1) and its integration and reduction, there will result,

$E D_5 = 1.3679 P_1 + 1.0717 P_2 + 0.7755 P_3 + 0.4793 P_4 + 0.1831 P_5$;

while in the triangular profile No. 2, with $b = 0.40$ we have

$E D_5 = 2.4141 P_1 + 1.8931 P_2 + 1.3720 P_3 + 0.8509 P_4 + 0.3298 P_5$.

It is next necessary to modify the results derived for the beam of

Mr. Shirreffs. constant thickness to suit the case of the beam comprised between radial planes 1 ft. apart at the up-stream edge of the dam. It will be readily seen that the elementary deflections will vary inversely as the moments of inertia of the rectangle having the height, l, and a base of 1.0 and a trapezoid having the same height and a base = 1.0, but a top, $c = 1 - \frac{l}{R}$, l being the joint length and R the radius of the up-stream face of the dam. The ratio of these moments of inertia is

$$m = \frac{c^2 + 4\,c + 1}{3\,(c + 1)}.$$

To introduce this value of I in the equations for $d\,\Delta$ and d tan. α would result in forms very difficult if not impossible of integration. The obvious approximation is to multiply the individual deflections and deflection angles by the reciprocal of m obtained from the joint length at the middle of the section under consideration, before cumulating the individual results for the deflections, D. A test computation shows that the error of this approximation is less than half of 1% at the base of Dam No. 1. It will diminish, of course, toward the top, while it will be somewhat greater for a lesser radius.

Before leaving this branch of the subject it may be remarked that we have, for the triangular profile, the following simple formulas, which aid rapid computation. Let V be the volume and W the weight of a vertical slice of the dam contained between radial planes 1 ft. apart at the up-stream face, u the distance of its center of gravity from the same face, v the distance from the vertical through the center of gravity to the point of the base where the resultant pressure strikes, M_o the total moment of all loads above the base, $\delta\,\gamma$ the unit weight of the masonry, and for the remainder the notation already used.

Therefore:

$$V = \frac{3\,R - b\,h}{6\,R}\,b\,h^2;\quad W = \delta\,y\,V;$$

$$\frac{u}{l} = \frac{2\,R - b\,h}{6\,R - 2\,b\,h} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$\frac{v}{l} = \frac{6\,M_o}{\delta\,y\left(3 - \frac{b\,h}{R}\right)b^2\,h^3} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Second.—The error resulting from computing the arch deflections as due to an arch having the radius of the extrados, instead of using the radius of the axis, will be corrected in the general solution of the deflections of the curved beam, which follows.

Third.—The following is offered as a solution of the problem of obtaining the movements and stresses in an arch subjected to a uniform normal or radial load. As the resulting equations are very much simpler and easier of application than the equations for the circular

arch, under whatever load, to be found in the textbooks, the methods and results are given in some detail. Mr. Shirreffs.

Imagine the segmental arch, of which 1–2′–3′ (Fig. 21) is one-half in its unloaded position, to be a portion of a "closed ring," or, what is equivalent, imagine the abutments upon which the arch rests to be frictionless, then, under the unit loading, q, producing a shortening, $2\,p$, of the original length, $2\,L$, the radius of the arch will be reduced by a proportionate amount, f, and the half arch will move to the new position, 4–5–6, in which position it will be in perfect equilibrium, with axial stresses at all points $= q\,R$, but without internal moments.

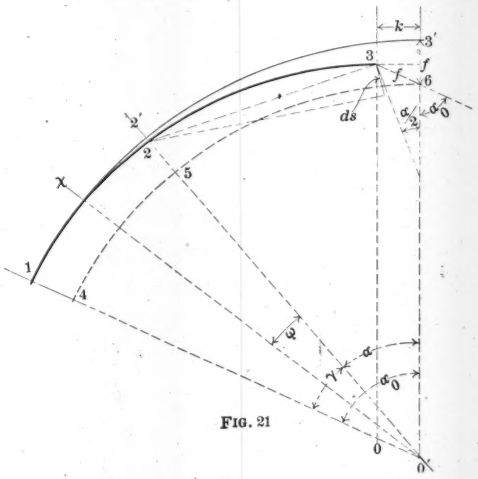


FIG. 21

Now imagine the arch to be severed at the crown, the right half being replaced by a crown thrust $= q\,R$, and the half arch then moved outward until Point 4 again coincides with Point 1, but without disturbing the perpendicularity of the axis at the skewback. The center of the arch will now be at 0 instead of 0′; Point 6 will be in the position of Point 3, at the horizontal distance, k, from the vertical through the middle of the span and at the vertical distance, Δ_c, below the original crown of the arch, but the crown joint will still be vertical. The arch will be assumed to have a width, l, and a thickness, 1. Since $f : \lambda = R : L$, and since $\lambda = \frac{q\,R}{A}\,\frac{L}{E}$, we will have,

$$k = f \sin.\ \alpha_o = \frac{q\,R^2}{E\,l} \sin.\ \alpha_o \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

and

$$\Delta_c = f\,(1 - \cos.\ \alpha_o) = \frac{q\,R^2}{E\,l}\,(1 - \cos.\ \alpha_o) \ldots\ldots\ldots\ldots (6)$$

Mr. Shirreffs.

At a point distant α degrees from the crown we shall have $\Delta_a : \Delta_c = \alpha_o - \alpha : \alpha_o$, and, therefore,

$$\Delta_a = \frac{q\,R^2}{E\,I}\,\frac{\alpha_o - \alpha}{\alpha_o}\,(1 - \text{cos. } \alpha_o) \ldots\ldots\ldots\ldots\ldots\ldots (6a)$$

In order now that the integrity of the arch be restored, it is necessary:

First, that the crown thrust, $q\,R$, be so diminished that, under the joint action of such diminished thrust and the loads on the half arch, the curved beam, 1-2-3, considered as fixed at the abutment, shall be deflected through the horizontal distance, k; and

Second, that the crown joint, which will have been deflected by this movement through the angle, β, shall be again made vertical in its new position, which can only be accomplished by the application thereto of a moment, M_c. As the original crown thrust, $q\,R$, just holds the arch in equilibrium against the action of the loads, the first movement will be identical with that caused by the action of a force, H', applied at the crown, equal to the necessary diminution of $q\,R$, and acting, therefore, toward the right. The total movement of the arch at any point will be obtained by combining the movements produced by H' and M_c with those resulting from the axial stress.

Neglecting the slight reduction in the compression of the arch occasioned by diminishing $q\,R$ by the amount, H', to obtain the value of H' we must substitute in the general expression for the deflection of a beam, $d\,s = M\,x\,\frac{d\,x}{E\,I}$ (moments taken about Point 2), $H'R(1 - \text{cos. } \alpha)$ for M; $2\,R$ sin. $\frac{\alpha}{2}$ for x; $R\,d\,\alpha$ for $d\,x$ and $\frac{l^3}{12}$ for I, and we get:

$$d\,s' = \frac{24\,R^3\,H'}{E\,l^3}\,(1 - \text{cos. } \alpha)\,\text{sin. } \frac{\alpha}{2}\,d\,\alpha.$$

But the horizontal component of this arc is $d\,k = d\,s'$ sin. $\frac{\alpha}{2}$, therefore, $d\,k = \frac{12\,R^3}{E\,l^3}\,H'\,(1 - \text{cos. } \alpha)^2\,d\,\alpha$. Integrating this expression, between the limits, α_o and 0, and equating the result with the value of k in Equation 5, we obtain:

$$H'\,R = \frac{q\,l^2}{12}\,\frac{2\,\text{sin. } \alpha_o}{3\,\alpha_o + \text{sin. } \alpha_o\,\text{cos. } \alpha_o - 4\,\text{sin. } \alpha_o} \ldots\ldots\ldots\ldots (7)$$

Now the moment, M_c, must cause the same angular deflection in the whole beam as the force, H'. In the general equation for angular deflection, $d\,\beta = \frac{d\,s}{x} = M\,\frac{d\,x}{E\,I}$, we must substitute, in one case, $H'\,R$ $(1 - \text{cos. } \alpha)$, and in the other, M_c for M, and in both cases, $R\,d\,\alpha$ for $d\,x$, thus getting,

$$d\,\beta = \frac{12\,H'\,R^2}{E\,l^3}\,(1 - \text{cos. } \alpha)\,d\,\alpha = \frac{12\,M_c\,R}{E\,l^3}\,d\,\alpha.$$

Mr. Shirreffs.

Integrating both expressions for $d\,\beta$, between the limits, α_o and α, and equating the results, we have:

$$M_c = H'\,R\,\frac{\alpha_o - \text{sin. } \alpha_o}{\alpha_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

This can be readily combined with Equation 7, and the result is:

$$M_c = \frac{q\,l^2}{12}\,\frac{\alpha_o - \text{sin. } \alpha_o}{\alpha_o}\,\frac{2 \text{ sin. } \alpha_o}{3\,\alpha_o + \text{sin. } \alpha_o \text{ cos. } \alpha_o - 4 \text{ sin. } \alpha_o} \ldots\ldots (8a)$$

To obtain the movement of the arch under the combined action of H' and M_c, we have to substitute again in the general equation $d\,s = M\,x\,\frac{d\,x}{E\,I}$ (moments referred to Point X, movement of Point 2) in one case, $H'\,R\,[1 - \text{cos. } (\alpha + \phi)]$, and in the other, M_c for M; and in both cases, $2\,R \text{ sin. } \frac{\phi}{2}$ for x, and $R\,d\,\phi$ for $d\,x$; and remembering that the movement produced by M_c is in the opposite direction to that produced by H', we have:

$$d\,s_a = \frac{24\,R^2}{E\,l^3}\,(H'\,R\,[1 - \text{cos. } (\alpha + \phi)] - M_c) \text{ sin. } \frac{\phi}{2}\,d\,\phi.$$

The radial component of this movement (the arch deflection which in a curved dam will produce stresses in the vertical beams) is $d\,D'_a = d\,s'_a \text{ cos. } \frac{\phi}{2}$, therefore,

$$d\,D'_a = \frac{12\,R^2}{E\,l^3}\,(H'\,R\,[1 - \text{cos. } (\alpha + \phi)] - M_c) \text{ sin. } \phi\,d\,\phi.$$

Integrating this between the limits $\phi = (\alpha_o - \alpha) = \gamma$ and $\phi = 0$, and substituting the values of $H'\,R$ and M_c, as derived from Equations 7 and 8, we get

$$d\,D'_a = C\,\frac{q\,R^2}{E\,l} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which C has the general value given in Equation 9*a*

$$C = \frac{2 \text{ sin. } \alpha_o}{\alpha_o}\,(1 - \text{cos. } \gamma) + \frac{\text{sin. } \alpha}{2}\,(2\,\gamma - \text{sin. } 2\,\gamma)$$
$$+ \frac{\text{cos. } \alpha}{2}\,(\text{cos. } 2\,\gamma - 1) \div \frac{3\,\alpha_o}{\text{sin. } \alpha_o} + \text{cos. } \alpha_o - 4 \ldots\ldots\ldots (9a)$$

When $\alpha = 0$, and therefore $\gamma = \alpha_o$, the value of C is in simpler terms,

$$C_c = \frac{\frac{2 \text{ sin. } \alpha_o}{\alpha_o}\,(1 - \text{cos. } \alpha_o) + \text{cos. } 2\,\alpha_o - 1}{\frac{3\,\alpha_o}{\text{sin. } \alpha_o} + \text{cos. } \alpha_o - 4} \quad \ldots\ldots\ldots\ldots\ldots (9b)$$

and again, for the semicircle, since then $\alpha_o = \frac{\pi}{2}$,

$$C_c = \frac{\frac{4}{\pi} - 1}{\frac{3\,\pi}{2} - 4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9c)$$

Mr. Shirreffs.

The diagram, Fig. 22, prepared in accordance with the foregoing formulas, will enable the coefficient $C\,C_c$ for the total crown deflection (embracing both the effect of H' and M_c, and the axial stress), or $D_c = \frac{C\,C_c\,q\,R^2}{E\,I}$, to be obtained by inspection, for an arch of any angle, with reasonable accuracy. The form of Equation 9 does not permit the general value of $C\,C_a$ to be diagrammed, except for a fixed value of α_o.

It should be remarked here that, as all the deductions have been based upon the assumption that the axis of the arch is loaded with q

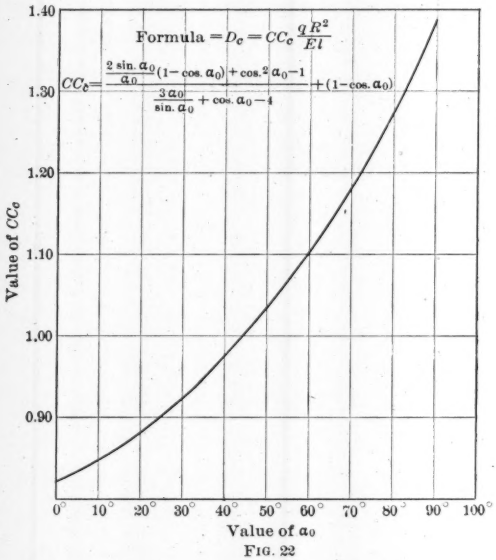


FIG. 22

per unit of length, it is necessary to insert in all the equations, not the value of q as it exists at the extrados of the arch, but this value multiplied by the ratio of the radius of the extrados to the radius of the axis. It may also be noted that if it be desired to obtain the stresses at any point of the arch, it will usually be close enough to combine with the axial stress, $q\,R$, the stresses of compression or tension produced by the moment, M_a. The force, H', may usually be neglected, as it has been in deriving the above formulas, as it is quite small in comparison to $q\,R$, although in very flat arches it will be necessary to take the effect of H' into account. If it be considered

necessary to embrace the effect of H' upon the arch compression in the formulas, they may become considerably more complex, but will yield then smaller values of H', and therefore of D, than the foregoing. The moment at any point, α degrees from the crown, is $H' R (1 - \text{cos. } \alpha) - M_c$, or Mr. Shirrefs.

$$M_a = \frac{q l^2}{6} \frac{\frac{\text{sin. } \alpha_o}{\alpha_o} - \text{cos. } \alpha}{\frac{3 \alpha_o}{\text{sin. } \alpha_o} + \text{cos. } \alpha_o - 4} \quad \ldots\ldots\ldots\ldots (10)$$

M_a becomes 0, or there is a point of contrary flexure in the curved beam used as an arch under a uniform radial load, when cos. $\alpha = \frac{\text{sin. } \alpha_o}{\alpha_o}$, or in the semicircle when cos. $\alpha = \frac{2}{\pi}$, or at about 50° from the crown.

Mr. Woodard's formula for obtaining the crown deflections, reduced to the notation thus far used herein, is,

$$D_c = \frac{q R^2}{E l} \alpha_o \frac{\text{cot. } \alpha_o}{2}, = C''_c \frac{q R^2}{E l},$$

and he further assumes that intermediate deflections will be proportional to the angular distance from the crown. Table 7 compares the deflections thus obtained in an arch having $\alpha_o = 74°$ (Arch No. 1 in Profile No. 2 in Table 6) with the deflections of the same arch considered as a curved beam, the coefficients requiring to be multiplied by $\frac{q R^2}{E l}$ in both cases to obtain the actual deflections.

TABLE 9.

$\alpha =$	0°	14° 48′	29° 36′	44° 24′	59° 12′
CC	1.228	0.833	0.722	0.483	0.211
Ratios to CC_c	1.000	0.678	0.588	0.394	0.172
CC''	1.714	1.371	1.028	0.686	0.343
Ratios to CC''_c	1.000	0.800	0.600	0.400	0.200
Ratio $\frac{C\,C}{C\,C''}$	0.72	0.61	0.70	0.70	0.61

It will be seen that, although the deflections of the curved beam are at all points much less than those resulting from the approximate method, there is no great departure from proportionality except near the crown. It will be interesting, also, to note the division of the total load between the arches and an intermediate beam. The following figures refer to the beam in Profile No. 2, which is situated 29° 36′ from the crown, the notation being that used before.

Joint.	1	2	3	4
Q	63 280	112 500	168 750	.....
P	21 420	74 900	145 210	.....
$\frac{u+v}{l}$	.....	0.577	0.743	0.872

Mr. Shirreffs.

Comparing these figures with those given in the latter part of Table 6, it will be seen that the intermediate beams carry a greater proportion of the load than the middle beam; in other words, the deflections of the shorter beams are reduced more rapidly than the deflections of the arches. This change in the condition of the arch load from perfect uniformity, of course, will have a tendency toward a readjustment of all deflections and loads. It would have been interesting, to the writer at least, if time had permitted the working out of a profile with a practical top width and also such as to avoid tension at all points, but this has been impossible.

In all that precedes, the investigation of the problem of the curved dam has been conducted upon the supposition that the various arch slices, into which the dam is divided for calculation of movements and stresses, are perfectly free to move relatively to each other, and that only as a part of each became in turn a part of some vertical beam would there be a composite action of the masonry of the dam. But the several arch slices are not free to move relatively to each other; they react on each other in such a way as to prevent the full shortening under the axial stress due to the load which is actually carried by them. This third composite action of the masonry virtually brings into play a second set of vertical beams, resisting movement this time in the direction of the axis of the dam, instead of transversely to this axis, and resisting it more effectually, too, than the transverse beam can, because each axial beam, in the dam built in a triangular valley, is reacted upon by a beam of diminished height and therefore of greater power of resistance. But even in the case of a dam the bottom of which should be level throughout until the common vertical abutment of all the component arches was reached, a case scarcely to be met in practice, the axial beams would still exert a restraining force on the movement of the upper arch slices. The effect of this second set of vertical beams must be to transfer the effect as to average axial compression, and therefore as to deflection of each upper arch slice, of some portion of the load on this upper slice to a lower one, but at a point nearer the final point of support. In other words, every load upon a curved dam is divided into three parts, one part passing through a horizontal arch to its abutment, another part by the aid of the vertical axial beams into the abutment of some lower arch slice, and the third, and probably in an arch of moderate radius, at least as regards the loads near the top, much the smallest part, through the transverse vertical beams into their foundation.

While it may not be possible to submit this very complicated action of the several resistances of the masonry of the curved dam to a satisfactory mathematical analysis, there can be no doubt that the resistance last referred to plays a very important part in the stability

of such structures. In the writer's opinion, it accounts entirely for the existence at the present time of the Zola Dam in France and the Bear Valley Dam in the United States. The former, built in 1843 and curved to a radius of 158 ft., has a height of 120 ft. and a base of 43 ft. ($b = 0.36$), while the latter, built in 1884, is curved to a radius of 335 ft., and is only 8.42 ft. thick at a point 44 ft. below the water line ($b = 0.18$). In the latter structure, especially, computation by the method herein first discussed would undoubtedly show the existence of large tensile stresses over probably the entire base. While the consideration of the axial vertical beams may not warrant the adoption of a profile which shows such stresses when thus computed, it should certainly inspire confidence in one free from tension when thus analyzed.* Mr. Shirreffs

As to the effect of temperature changes, unfortunately, there are few data with which to work. There is for instance an almost entire lack of information as to the penetration into the body of a massive masonry structure of the effect of the exterior seasonal changes. With the facility afforded by electrical methods for measuring such effects, it is to be desired that every engineer who builds such a structure at the present time should make the inexpensive preparation required to observe in this manner the temperature changes in the interior of the mass. The writer believes that where a radius short enough to develop the economic advantage of the curved dam can be adopted (and this seems to require a radius not much greater than 400 ft.), there need be no anticipation that temperature changes will operate to the injury of the structure.

It may be of interest to call attention to the only case, within the writer's knowledge, where the movements of a masonry dam have been actually observed. The case is that of the dam at Remscheid, Germany, built by Professor Intze, the results of such measurements being reported by him in the *Zeitschrift des Vereines deutscher Ingenieure* for 1895. A brief abstract of this article was given in *Engineering News* of January 30th, 1896, and the following is quoted therefrom:

"Careful measurements have been made to ascertain the radial movements * * * due to variations in pressure and temperature. The former have caused the greatest movement at the center—$1\frac{1}{16}$ ins. —while at symmetrical points between the center and the wings the observed movement was $\frac{9}{16}$ in. The greatest effect due to temperature changes was observed during a very hot and dry summer upon the curved surface, and was $\frac{7}{8}$ in. on one side of the dam against only $\frac{1}{8}$ in. on the other. This unsymmetrical action is explained by the fact that the former was entirely exposed to while the latter was largely protected against the influence of the sun. The author believes that if the dam had not been curved these movements would certainly have produced cracks."

* The above was written before reading the remarks of Mr. Frizell, p. 135, in discussing the same point. It is allowed to stand, as at least emphasizing, and perhaps making a little clearer, the action to which Mr. Frizell calls attention.

Mr. Shirreffs. It may be added that the dam was about 13 ft. wide at the top and 49 ft. wide at the bottom where the height was greatest (about 82 ft.), the down-stream face curving vertically between these points. The horizontal radius of the up-stream face at the top was 410 ft. It is interesting to compare the actual movement of this comparatively low dam with the deflection of only $\frac{1}{8}$ in. which Mr. Woodard computes for the Lake Cheesman Dam, assuming $E = 3\,000\,000$ lb. per sq. in.

In concluding, the writer desires to acknowledge the valuable assistance rendered him by Mr. F. F. Moore in verifying many of the deductions and computations. This assistance was the more appreciated because it was rendered through love for his profession.

Mr. Wisner. GEORGE Y. WISNER, M. AM. SOC. C. E. (by letter).—The work discussed in this paper is particularly interesting at the present time, for the reason that a number of dams are likely to be constructed in the near future, under somewhat similar natural conditions, for the storage of flood waters for irrigation purposes in the West; and especially as the structure, as completed under the present Chief Engineer, A. E. Kastl, M. Am. Soc. C. E., is one of which the engineers who designed and supervised the work, and the contractors who constructed it, may well feel proud.

Many of the large undertakings of recent years have been criticized so severely for faulty construction, both as to design and workmanship, that it is refreshing to have an actual example of completed work in every way superior to that called for in the plans and specifications under which it was built.

In regard to the rock-fill dam, originally proposed for this project, the structure would doubtless have answered the purpose for which designed if it could have been completed before being subjected to flood overflow, but, from a professional point of view, the dam, as completed, is better designed to meet the natural conditions and to give satisfactory results.

Knowing the inevitable result of permitting a heavy flood to flow over the unprotected top of a partially completed rock-fill dam, it is somewhat surprising that greater precautions were not taken.

In the case of a rock and earth-fill dam, 50 ft. high, soon to be constructed in the Snake River, in Idaho, the plans provide for a by-pass around one end of the dam of such dimensions as will take care of the flow during construction. This by-pass, afterward, will be closed by regulating gates, for emptying the reservoir, and a thin, reinforced concrete, arched dam resting on the gate piers and abutting against the side walls of the by-pass.

It is stated in the paper that a balance valve was placed at the entrance of the lower tunnel, for the purpose of regulating the outflow, of which valve no description is given.

It is a well-known fact that many of the devices known as balance

valves do not work as such in actual practice, and, as the writer is informed that the one in question has never been operated, it would be interesting to know its construction, and whether it is likely to meet the requirements. Mr. Wisner.

It is inferred from the statements of the authors that the arch type of dam was adopted for the reason that such type was better adapted for the site than a straight dam, and was no more expensive, rather than from any expected additional stability to be derived from such form. However, it appears from the very interesting analysis of the stresses in the dam, given by Mr. Woodard, that at the top of the dam the arch carries nearly half the load, 6% half way down and nothing at the bottom.

When it is considered that the lower 100 ft. of this dam is practically an immense wedge of masonry held firmly in position by the solid granite of the canyon walls, preventing any tendency to slide or overturn, it is probable that this arch really takes care of a much larger percentage of the stress, which would otherwise be transmitted to the foundation, than shown by the analysis.

It is assumed in the analysis that the modulus of elasticity is the same for the vertical sections of the dam as in the horizontal rings used in computing the pressures transmitted to the side walls through the arch. In the mid-stream vertical sections of the dam, the concrete and granite boulder masonry constitutes the greater portion, while, for the upper third of the dam, where the arch is most effective, the granite masonry faces greatly increase the average modulus over that of the vertical section. The writer is strongly of the opinion that an analysis of the probable stresses in the structure, made with approximately correct moduli for the vertical and horizontal sections, would show the stability to be much greater than stated by the authors.

In localities having as wide a range of temperature as exists in most of the Western States, the stresses developed in dams from changes of temperature may, under certain conditions, exceed those arising from the water pressure on the face of the dam; and, where reservoirs are likely to remain only partly full for long periods, the use of steel in the upper portion of the structure is a precaution worthy of careful consideration. In the case of a straight dam, the tensile strains from change of temperature may reduce the stability to that of the theoretical section generally used in designs for gravity sections, which, with the uncertain characteristics of the materials used, cannot be considered safe. In the case of the proposed Salt River Dam, in Arizona, the upper 100 ft. of the dam will be reinforced with steel, and, as an additional precaution, the masonry of that portion of the dam will be put in only when the temperature is below the average for the year, thereby insuring either normal or compressive stresses for most of the time.

Mr. Wisner. During the excessively cold weather of the winter of 1903–04, a vertical temperature crack, some 30 ft. long and ⅛ in. wide, breaking directly through the solid granite face stones, developed in the Cheesman Dam, which closed and nearly disappeared when the weather moderated. In a straight dam, with a full reservoir, such a crack would have been likely to develop leaks and, possibly, have endangered the structure, but, in the arch dam at Cheesman, it did no practical damage whatever, the crack only extending a short distance from the face.

Taking into consideration the natural conditions at the site of the Cheesman Dam and the design adopted, it is probable that a mid-stream cross-section with a base two-thirds of that used, would have been absolutely safe—in fact, the only indication of seepage that shows, with 100 ft. of water against the face of the dam, is through the narrow granite point against which the north end of the dam abuts.

In the arch type dam, such as that described, it is probably true that the rigidity of the structure will prevent it from acting as a theoretical arch, but, since the transmission of stresses through the structure, either vertically or horizontally, pre-supposes distortion, it is practically self-evident that the stresses from water pressure on the face of the dam will be distributed to the side walls by the arch, as well as to the foundation by the bending movement in the mid-stream vertical section—the amount of which will depend upon the design of the dam and the nature of the materials of which it is constructed.

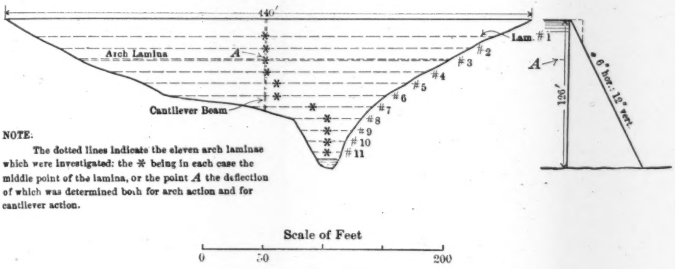
The substitution of a granite masonry spillway for one in the natural rock of the granite point at the north end of the dam has strengthened the structure and improved its appearance. The completed dam is probably the best example of modern dam construction in the United States, and is well worthy of a visit by engineers interested in work of this class.

Mr. Duryea. Edwin Duryea, Jr., M. Am. Soc. C. E. (by letter).—About a year ago the writer had occasion to design a masonry dam for a site especially favorable to the arch type. Economy made this type desirable, while the presence in the same State (California) of such successful examples as the Bear Valley, Sweetwater and Upper Otay Dams made it especially advisable to use the arch type if it could be shown by investigation to be theoretically justifiable. An investigation, therefore, was made with some care, in order to furnish definite grounds for the acceptance or rejection of this type.

The structural plan investigated is shown by Figs. 23, 24 and 25, the principal dimensions being a length of 440 ft. on the crest, a maximum height of 126 ft., a vertical up-stream face having a radius of 200 ft., and the down-stream face battered 6 in. horizontally to 12 in.

vertically. This profile, if the entire water pressure is assumed to be carried by arch action alone, gives, throughout the dam, a nearly uniform arch stress of about 12½ tons per sq. ft. of arch section. Mr. Duryea.

The investigation was made on the same general lines as given in this paper, and, while not exhaustive, showed conclusively that, for the structure and conditions assumed, only a small proportion of the total water pressure could be borne by arch action until after a sliding or giving, to develop this action, had occurred on several horizontal planes, and especially at the base of the dam.



NOTE.

The dotted lines indicate the eleven arch laminæ which were investigated: the ✻ being in each case the middle point of the lamina, or the point *A* the deflection of which was determined both for arch action and for cantilever action.

Scale of Feet

0 50 200

FIGS. 23 AND 24.

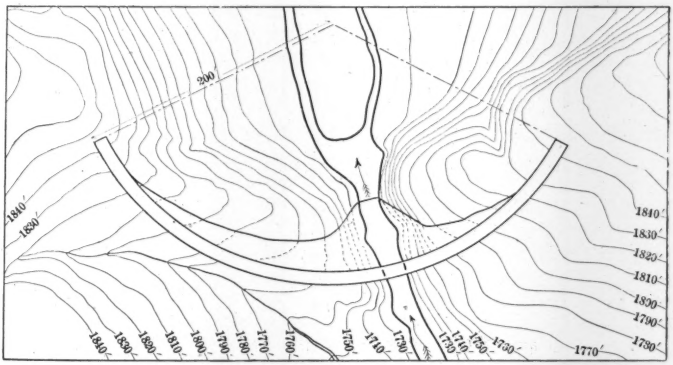


FIG. 25.

The computed proportions of the total water pressure which could be borne by arch action before such sliding occurred vary irregularly from about 16% near the crest of the dam to zero at the base, the average at twelve points equidistant vertically being about 8 per cent. These proportions are shown in detail in Fig. 29. They are only approximations, being in each case for the middle point only of the horizontal arch lamina in question, but, at least, they show conclusively that (unless sliding has occurred) only a small amount of

Mr. Duryea.

assistance in carrying the water pressure can be expected from the arch form. Reliance on arch action, as justifying a section thinner than a gravity section, therefore, was reluctantly abandoned.

It seems self-evident to the writer, however, that the gravity section should be built in an arch form whenever practicable, as a dam arched in plan will generally cost but little, if any, more than a straight dam of equal section, and, in case of a failure of the gravity section, the arch action will be developed and will in all probability prevent a collapse of the structure.

Rough approximate estimates of the cost of a dam at the site in question were made on five plans, as follows:

		Percentages.		
1.—Gravity type, arched in plan	$340 000	100	152	136
2.—Sweetwater section, arch type	230 000	68	103	92
3.—Arch type, as described	223 000	66	100	89
4.—Rock-fill dam	330 000	97	148	132
5.—Buttress type	250 000	74	112	100

The gravity type referred to is Wegmann's practical profile No. 2. The Sweetwater section is that of the Sweetwater Dam extended downward for a greater height by differences. The rock-fill dam is 10 ft. wide on the crest, with an up-stream slope of 1:1, and a down-stream slope of ¾ horizontal to 1 vertical. The buttress dam was composed of eight semi-circular arches supported by buttresses 50 ft. apart, center to center. The minimum thickness of these arches was 5 ft., increasing to 12 ft. at the lowest point of the dam by steps on the down-stream face, the up-stream face being vertical. The buttresses were 7 ft. thick at the top and increased by steps on both sides to 15 ft. thick at the base of the highest buttress.

In making these comparative estimates, the same price per cubic yard of masonry was assumed for Types Nos. 1 to 3, inclusive. The price per cubic yard assumed in estimating the rock-fill dam was one-quarter of that for Types Nos. 1 to 3; while the price assumed in connection with the buttress dam was 30% in excess of that used for Types Nos. 1 to 3. The high comparative cost of the rock-fill dam is due mainly to its bottom width being greater than the length of the narrow gorge forming the dam site, thus allowing a great spread in the base of the rock fill. The estimated cost of the buttress dam is relatively high because it was not practicable to locate it entirely on the narrow dike, as could be done with Types Nos. 1 to 3.

In all five types considered, floods were to be taken care of by a liberal spillway, east of the dam, through the natural rock.

From the five plans compared, the buttress type was chosen because of its comparatively low cost, in conjunction with the fact that strict analysis justifies the structure as a safe one. This plan was thoroughly investigated, and full stress sheets and drawings were prepared. The maximum allowable stress in the arches from water pres-

sure was fixed at 10 tons per sq. ft., and that in the buttresses, from a combination of water pressure and weight of masonry, at 15 tons per sq. ft. The maximum allowable shear in the buttresses was fixed at 5 tons per sq. ft. The buttresses were proportioned so as to keep the resultants of horizontal and vertical forces within the middle thirds. Mr. Duryea.

While the buttress design proposed for this dam lacks conservatism, in being higher than any buttress dam yet built, no other valid objection can be made to it except the possibility (a very remote one, even in California) of its injury by earthquakes. In such a climate, no possible injury can occur from the expansion or impact of ice.

In some locations, the writer believes that the buttress type will give a dam, not only much lower in cost, but, in addition, much safer, than will the Wegmann gravity section. This section seems to be generally regarded as safe without question. It makes no provision against uplift, however, and the possibility of uplift certainly often exists. At the site described by the writer the foundation is hard, sound rock, giving an opportunity for such a good junction with the bed-rock as would leave little chance of uplift. At this site the only practical advantage of a buttress over a gravity dam, therefore, would be its saving in cost. In another dam designed by the writer, however, about 30 ft. high and having to pass floods of possibly 15 ft. in depth over the full length of its crest, the foundation was a rather soft serpentine. In this case the writer believes that a buttress dam, with its practical freedom from the possibility of uplift, would be, not only much less costly than a Wegmann section, but also much safer. In this buttress design the up-stream face was made sloping, 6.4 vertical to 10 horizontal, with the down-stream ends of the buttresses vertical. It was proposed to strengthen both the curtain and the buttresses with expanded metal and rods, though, in the computations for strength, no account was taken of this metal.

In the following are given the assumptions and formulas by which were computed the proportions of water pressure which could be borne by the arch action of the structures shown in Figs. 23, 24 and 25. The formulas were developed and the computations made by C. B. Wing, Assoc. M. Am. Soc. C. E., who also made the computations and drawings for the two buttress designs mentioned.

In a masonry dam, any point, as *A*, Fig. 23, is deflected down stream as the water is allowed to rise to the top. In an arch-type dam, the displacement of the point, *A*, corresponds at the same time to the deflection of a cantilever beam of unit length along the dam between two transverse vertical sections, and to the deflection of an arch lamina of unit height of the dam between two horizontal planes. The deflection as a cantilever beam is caused by the bending moments induced by that portion of the water pressure carried by cantilever action, and the deflection as an arch is due to the arch contraction caused by the uniform thrust induced by that portion of the water

Mr. Duryea. pressure borne by arch action. Such a vertical cantilever beam and horizontal arch lamina are shown by dotted lines in Fig. 23.

The total water pressure on the dam will be shared between the two systems, arch and cantilever, directly in proportion to their respective rigidities, or inversely as their deflections. If, for any point, A, of an assumed dam section, equations be developed (1) for the deflection of the vertical cantilever beam, and (2) of the horizontal arch lamina, in each case in terms of the total unit water pressure (or weight per cubic foot of water), a relation between the proportions carried in each way will be obtained, and the amount of each proportion may be determined. The computations, at best, were very complex and tedious, and, for simplicity, the points, A, were taken at the centers only of the corresponding arch laminæ. The method used could have been applied to other points of each arch lamina, but the computations would have been still more complex, and, for the information sought, it seemed unnecessary to do so.

Assumptions.—The cantilever beams were assumed to be fixed in direction at the base, with planes before flexure still planes after flexure. The arch laminæ were assumed to be two-hinged, or changeable in direction at each bank. These two assumptions are contradictory, and both tend to reduce the apparent proportion of pressure carried by arch action. They were used for simplicity, however, and as being sufficiently exact for the end sought.

Nomenclature.—(See Figs. 26 and 27).

1.—Vertical section through the point, A:

Let h = height of dam from base (below A) to crest;
b = thickness of dam at base (below A),
x = height from base of dam to any horizontal section;
f = thickness of dam at height, $x = \frac{1}{2}(h - x)$.

2.—Horizontal section through the point, A:

Let s = span of horizontal arch lamina before being shortened by pressure;
r = mean radius of horizontal arch lamina before being shortened by pressure;
l_1 = chord of half of horizontal arch lamina before pressure is applied;
l_2 = chord of half of horizontal arch lamina after pressure is applied;

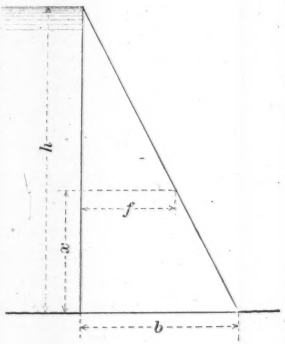


FIG. 26

Mr. Duryea.

T = thrust on horizontal arch lamina due to proportion of water pressure borne by it;

e = total shortening of chord, l_1, due to the thrust, T;

y_1 and y_2 = ordinates corresponding to l_1 and l_2;

$d = y_1 - y_2$ = deflection of the point, A.

3.—Water pressures:

Let k = the weight of a cubic foot of water = 62.5 lb.;

k_g = proportion of the unit weight of water (number of 62.5ths) which will be borne by gravity action;

k_a = proportion of the unit weight of water (number of 62.5ths) which will be borne by arch action.

4.—Compression of masonry:

Let E = modulus of elasticity of masonry = 1 500 000 lb. per sq. in.

The value of the modulus of elasticity of masonry is very variable and uncertain, and, therefore, it would be much better, theoretically, to eliminate this quantity from the final equation. This was not done, however, but it appears in both terms of the equation, and its effect, as will be seen later, is thus practically eliminated.

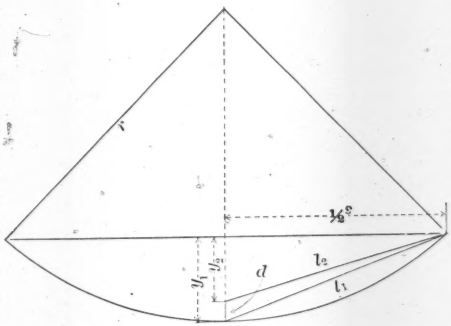


FIG. 27.

Development of Equation.—By the ordinary theory of flexure, the deflection, d, of any point, A, in a vertical section of the dam of unit length (along the crest), acting as a cantilever beam and subjected to that proportion of the total water pressure which can be borne by cantilever action, will be given by the formula,

$$d = \frac{h^3}{b^3} \frac{k_g}{E} x^2,$$

or

$$\frac{8\, k_g}{E} x^2$$

for the section under consideration.

This formula is derived as follows: While, theoretically, the formula may not be exactly correct, the results obtained from it must be a very close approximation.

Mr. Duryea.

The differential equation of the elastic curve is

$$\frac{d^2 y}{d x^2} = \frac{M}{E I},$$

where M = moment of the external forces;

and I = moment of inertia of the section of beam.

The cantilever beam is formed of a transverse section of the dam, of unit length (along the crest), and is shown, with the water pressure acting upon it, in Fig. 28.

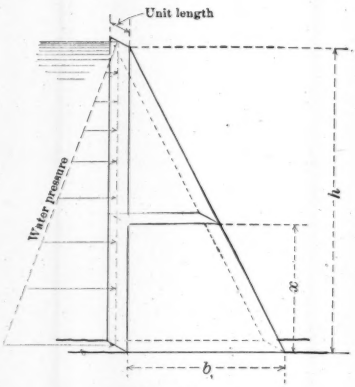


FIG. 28.

The moment of the forces acting above any section, x linear units from the bottom, is

$$M = \frac{(h-x)\,k}{2}(h-x)\frac{h-x}{3} = \frac{(h-x)^3}{6} - k,$$

where k = the weight of a cubic unit of water.

The moment of inertia of the rectangular horizontal section at x is

$$I = \frac{\text{breadth} \times \text{height}^3}{12}, \text{ or, as the breadth is unity,}$$

$$I = \frac{\text{height}^3}{12}$$

$$= \frac{\left(\frac{h-x}{h}\,b_1\right)^3}{12} = \frac{{b_1}^3}{12\,h^3}(h-x)^3,$$

and

$$\frac{d^2 y}{d x^2} = \frac{M}{E I} = \frac{12\,(h-x)^3\,k\,h^3}{6\,(h-x)^3\,{b_1}^3\,E} = \frac{2\,h^3\,k}{{b_1}^3\,E}.$$

Integrating,

$$\frac{d y}{d x} = \frac{2\,h^3\,k}{{b_1}^3\,E}x + (C_1 = 0),$$

and again,

$$y = \frac{h^3\,k}{{b_1}^3\,E}x^2 + (C_2 = 0),$$

and finally, as $h = 2\,b_1$,

$$v = \frac{8\,k}{E}x^2.$$

Replacing y by the symbol for deflection, d, and k by k_g, the proportion of the water pressure borne by cantilever action, the equation becomes,

$$d = \frac{8\,k_g}{E}x^2, \text{ as given.}$$

Mr. Duryea.

The deflection of the horizontal arch lamina at the point, A, is more difficult to obtain. An approximate determination of the deflection at the center of a flat arch may be made as follows:

Assume the deflection, d, to be due to a decrease in the length of the chord, l_1 (Fig. 27), the length becoming l_2 after the load is applied;

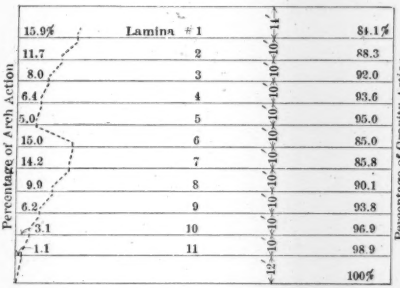


FIG. 29.

$$d = y_1 - y_2,$$

$$y_1 = r - \sqrt{r^2 - \frac{s^2}{4}};$$

let $$\sqrt{r^2 - \frac{s^2}{4}} = a,$$

then $$y_1 = r - a$$

$$l_1 = \sqrt{y_1^2 + \frac{s^2}{4}} = \sqrt{r^2 - 2\,r\,a + a^2 + \frac{s^2}{4}}$$

$$l_2 = l_1 - e,$$

and, since the arch lamina is of unit thickness vertically,

$$e = \frac{T}{f\,E}\,l.$$

The thrust or compressive force in any arch lamina, x, will be caused by the water pressure, $k_a\,(h - x)$, at that lamina, and will be

$$T = k_a\,(h - x)\,r.$$

Therefore, $$e = \frac{k_a\,(h - x)\,r}{f\,E}\,l;$$

and $$l_2 = l_1\left(1 - \frac{k_a\,(h - x)\,r}{f\,E}\right);$$

let $$\left(1 - \frac{k_a\,(h - x)\,r}{f\,E}\right) = c,$$

then $$l_2 = l_1\,c.$$

$$y_2 = \sqrt{l_2^2 - \frac{s^2}{4}} = \sqrt{l_1^2\,c^2 - \frac{s^2}{4}}$$

$$= \sqrt{\left(r^2 - 2\,r\,a + a^2 + \frac{s^2}{4}\right)c^2 - \frac{s^2}{4}}$$

The point, A, is a material point and can have but a single deflection at any one time. The deflection in the arch lamina at this point, therefore, must necessarily be the same as that in the cantilever

Mr. Duryea. beam. Then,

$$d = y_1 - y_2 = \frac{8\,k_g}{E}\,x^2$$

and substituting values of y_1 and y_2,

$$(r-a)-\sqrt{\left(r^2-2\,r\,a+a^2+\frac{s^2}{4}\right)c^2-\frac{s^2}{4}}=\frac{8\,k_g}{E}\,x^2.$$

Substituting in this equation the values of a and c, and replacing k_g by its equivalent, $(k - k_a)$, or $(62.5 - k_a)$, the equation is as follows:

$$r-\sqrt{r^2-\frac{s^2}{4}}-\sqrt{\left[r^2-2\,r\sqrt{r^2-\frac{s^2}{4}}+\left(r^2-\frac{s^2}{4}\right)+\frac{s^2}{4}\right]\times\left[1-\frac{k_a\,(h-x)}{f\,E}\,r\right]^2-\frac{s^2}{4}}=\frac{8\,(62.5-k_a)}{E}x^2,$$

which gives, after simplification, as the final equation,

$$r-\sqrt{r^2-\frac{s^2}{4}}-\sqrt{\left[\frac{s^2}{4}+\left(r-\sqrt{r^2-\frac{s^2}{4}}\right)^2\right]\times\left[1-\frac{k_a\,(h-x)}{f\,E}\,r\right]^2-\frac{s^2}{4}}=\frac{500-8\,k_a}{E}\,x^2.$$

After the selection of any particular arch lamina, the only unknown quantity in this equation is k_a, or the number of pounds per cubic foot of water which can be borne by arch action.

TABLE 8.—DATA AND RESULTS.

Numbers of arch laminæ.	DATA, IN FEET.						RESULTS, IN 62.5THS, AND IN PERCENTAGES.		
	r	s	h	x	b	f	k_a	k	k_g
1	195	309	74	60	37	10	9.96 = 15.9%	62.5 = 100%	84.1%
2	194	275	74	50	37	12	*7.34 = 11.7	" "	88.3
3	191.5	241	74	40	37	17	5.00 = 8.0	" "	92.0
4	189	192	79	35	39.5	22	4.00 = 6.4	" "	93.6
5	186.5	146	84	30	42	27	3.1 = 5.0	" "	95.0
6	184	93	119	55	59.5	32	9.4 = 15.0	" "	85.0
7	181.5	67	126	52	63	37	*8.9 = 14.2	" "	85.8
8	179	53	126	42	63	42	6.2 = 9.9	" "	90.1
9	176.5	44	126	32	63	47	3.9 = 6.2	" "	93.8
10	174	34	126	22	63	52	1.95 = 3.1	" "	96.9
11	171.5	25	126	12	63	57	*0.67 = 1.1	" "	98.9

*By the longer method, k_a = 7.4, 8.8 and 0.06 for Laminæ Nos. 2, 7 and 11, respectively.

Mr. Duryea.

The solution of this equation involved a great amount of labor, as seven-place logarithmic tables were not sufficiently accurate to give correct results, and it was found necessary to make the computations by ordinary multiplication. The proportions borne by arch action were computed laboriously by this method for three arch laminæ, the data and results being given in Table 8. An extended investigation* of arch dams then came to the writer's attention, and, by a formula for arch deflection found therein, the proportions borne by arch action, in the cases of the three laminæ already investigated and in eight others, were computed in a few hours.

The formula in question may be found on page 82 of Vischer and Wagoner's paper, and is as follows:

$$e = d\ V,$$

in which

e = the total shortening under the arch pressure of half the arch lamina;

d = the deflection of the center point of the arch lamina;

V = one-quarter of the total angle (in terms of the radius) subtended by the arch lamina.

From the theory of compression of elastic bodies,

$$e = \frac{T}{f\ E}\ l = \frac{k_a\ (h - x)\ r}{f\ E}\ l,$$

where the vertical thickness of the arch lamina is unity and the nomenclature is as before, except that l now represents the half length of the arch lamina instead of the length of the half chord.

For the profile being investigated,

$$f = \tfrac{1}{2}\ (h - x)$$

and

$$e = \frac{2\ k_a\ r}{E}\ l.$$

Therefore,

$$d = \frac{e}{V} = \frac{2\ k_a\ r}{E\ V}\ l;$$

but

$$2\ V\ r = l, \text{ or } V = \frac{l}{2\ r}.$$

Therefore,

$$d = \frac{2\ k_a\ r}{E\ V}\ l = \frac{2\ k_a\ r}{E\ \frac{l}{2\ r}}\ l = \frac{4\ k_a\ r^2}{E},$$

as an expression for the deflection of the center point of an arch lamina.

As before, the deflection of the same point, in terms of the deflection of a vertical cantilever beam, is

$$d = \frac{8\ k_a}{E}\ x^2 = \frac{500 - 8\ k_a}{E}\ x^2.$$

The deflection of the arch lamina and that of the cantilever beam must be the same.

*Vischer and Wagoner, "On the Strains in Curved Masonry Dams," *Transactions*, Tech. Soc. Pacific Coast, Vol. VI, Dec., 1889, pp. 75–151.

Mr. Duryea.

Therefore,
$$\frac{4\,k\,r^2}{E} = \frac{500 - 8\,k_a}{E}\,x^2,$$

and
$$k_a\,r^2 = (125 - 2\,k_a)\,x^2 = 125\,x^2 - 2\,x^2\,k_a,$$
$$k_a\,(r^2 + 2\,x^2) = 125\,x^2,$$

and, finally,
$$k_a = \frac{125\,x^2}{r^2 + 2\,x^2}.$$

By this formula the proportion of the total water pressure carried by arch action was computed for eleven arch laminæ, and the results obtained are given in Table 8. The close correspondence between the values as computed by the two methods is notable, and seems to be a check, not only on the correctness of the methods and computations, but also to show that the choice (in connection with the longer method) of a value for the modulus of elasticity of masonry has but a small effect on the resulting computed proportion of arch action.

The method of Messrs. Vischer and Wagoner, of course, is much to be preferred, both theoretically and practically. Their valuable study of arch dams seems to be too little known to the profession.

Mr. Williams.

G. S. WILLIAMS, M. AM. SOC. C. E.—To the speaker, outside of the very interesting information in regard to the methods of construction and the details of the design of the Lake Cheesman Dam, the most valuable part of the paper is that by Mr. Woodard. Engineers have been waiting a good while for an American to attack this problem of the analysis of the relative strains set up in a dam which combines the arch and the gravity sections, and they should feel grateful to the authors for having brought out a solution of the problem. It may be remarked, however, that where a dam abuts against vertical walls the action becomes more nearly akin to that of a plate supported on three edges than to that of a beam fixed at one end.

In looking over the paper and the illustrations, other possible solutions of this problem have been suggested, and it may be well to mention two or three treatments which might have been introduced, perhaps with no better results, but at least they should be interesting.

Undoubtedly, all have been impressed with the fact that, to start with, there were two solid abutments, and, having that for a dam of moderate span, and by moderate is meant less than 400 or 500 ft., there seems to be no other type that combines greater stability with a minimum of material than an arch, pure and simple, with the gravity section eliminated.

An arch, to act as an arch, must be designed so as to have a considerable degree of elasticity, or the arch action will fail; that is to say, it is not possible to get arch action with very thick material unless the stresses are extremely high, and, therefore, the authors were fully justified in eliminating from consideration as an arch the lower section of the dam.

In casting up the possibilities at the authors' site, without going into the matter very definitely, it appears that the controlling point for

Mr. Williams.

an arch dam was at an elevation of about 150 ft., where, on account of the span, a radius of about 200 ft. would be necessary. Limiting the stresses to those computed by the authors for the existing structure, it appears that an arch of about 25 ft. thickness would be required, and, starting with that, it would decrease to 1 ft. or any greater thickness that was desired, at the top, without increasing the stresses beyond those which the authors allowed in the structure as they designed it. Their stress, as they show it, is about 240 lb. per sq. in., or 35 000 lb. per sq. ft., and, taking that as a limiting stress, which, all will agree, is a perfectly safe stress for masonry of this character, a dam might have been designed for this place having a maximum thickness of 25 ft., and from that reducing to any desired thickness at the top; that design being a simple vertical cylindrical arch above the 150-ft. contour, and being of smaller radius below.

Another solution that appears in this case, is to have designed the dam as an inverted cone, and the spot would have been quite favorable for such a treatment. It will be recalled that the thrust of an arch under normal loads is equal to the pressure on the extrados into the radius of the extrados—not that of the center line, as is often incorrectly assumed—so that, by varying the radius, the thickness, or the total thrust to be taken by the arch, may be varied. Starting in that way, it would be possible to make the dam of equal strength, but much less than 25 ft. in thickness at the base, if such were desired. Whether or not it would be safe involves consideration of the permeability of the masonry, and that is possibly an open question with some.

Still another treatment would have been to make the base of the dam a segment of a sphere, and, recalling that the thrust in a spherical dome is only one-half of that in a cylinder of equal radius, still less material might thus be used in the base of the dam, and then, as the upward thrust of the sphere would have to be absorbed by the weight of the material above it, the radius of the sphere would be limited to that giving a thrust not greater than the weight of the material above its equator.

These are only offered as possible solutions, and there may have been reasons, other than structural ones, for not adopting such departures from former general practice.

Not long ago it fell to the speaker's lot to design a dam for a spot which seemed to be equally well, if not even better, suited for the arch solution of the problem, and, as illustrating a purely arch design, that location, the structure designed, and the structure built, are shown in Plates IX, X and XI. This site was in the vicinity of Ithaca, N. Y., about 2 miles from the center of the town, and the work was designed for the Ithaca Water-Works Company.

The Six-Mile Creek Dam.

Location and Conditions.—Six-Mile Creek, a stream having a quite precipitous drainage area of about 48 sq. miles above the point in

Mr. Williams. question, there passes in a northerly direction through a gorge or miniature cañon about 500 ft. long and 90 ft. wide. The location selected for the dam, Fig. 1, Plate IX, was near the upper end of this gorge, where the rock on the east side rises to a height of 90 ft. above the bed of the stream, overhanging in its rise 4 or 5 ft., and on the west side a similar wall, receding 6 ft. in its height, rises 70 ft. above the bed. On both sides, the rock was surmounted by a heavy deposit of drift clay, containing boulders, but quite impervious, and rising with a slope of nearly 30° for 50 or 100 ft. more. The rock was the bluish-gray shale, so common in that region, traversed at intervals of from a few inches to several feet with nearly parallel fissures, the sides of which, except near the exposed walls of the gorge, were in close contact, and, where open, the seams were filled with fine clay washed in from the covering beds. The planes of the fissures were also nearly parallel to the axis of the gorge. On the exposed faces the rock was weathered for a depth of about 6 in. to a varying extent, thereby showing very clearly its stratified character; but, where the weathered surface was removed, the faces of the fissures showed a smooth, dense rock without apparent horizontal seams, except at intervals, usually of several feet. The bottom of the gorge was covered to a depth of about 6 ft. by a deposit of sand and gravel, caused by the construction, a few years previously, of a small dam at its outlet. The bed itself was of shale rock, similarly fissured and nearly level throughout three-fourths of the width of the gorge, and rising in steps of about 4 ft. near the west wall.

As the location was only a short distance above the city, and, as a failure of the structure would involve considerable financial loss, not only to the citizens generally, but especially to the Water Company, whose pumping station was on the bank of the stream less than a mile below, it was at once apparent that a type of dam should be selected which would be stable against all possible contingencies.

The conditions being such as to call, first of all, for an overfall dam, and the seams in the bottom running longitudinally of the gorge and thus possibly permitting percolation and an upward pressure on the base, were to the speaker the strongest of arguments against the adoption of a gravity section of the ordinary type in this location.

Were the problem presented of carrying a roadway, even for the heaviest kind of traffic, across the gorge in question, no one would for a moment think of laying pipes or building culverts along the bottom and filling the chasm level full of masonry on top of them, nor would he put in a series of piers and connect them with short plate girders; but the one obvious and correct solution would be to span the depression with an arch either of metal, wood or masonry. Bearing in mind that the arch, under vertical moving loads, can never be in equi-

FIG. 1.—SITE OF SIX-MILE CREEK DAM.

FIG. 2.—BUILDING THE SIX-MILE CREEK DAM.

librium, but must always resist varying bending moments, and that under normally applied uniform forces a circular rib will be in equilibrium, and subject to no bending moments, except those possibly induced by temperature changes and the compression of the material itself, which are also similarly possible in arch bridges, the propriety of applying the concrete arch to the problem becomes at once apparent, for it will be seen that the only possible means of failure for a circular arch under normal uniform forces is by the ultimate crushing of the material; and the conditions of the permeability of the base or foundation rock and consequent upward pressure underneath the dam, or a side pressure at the ends, have no influence upon the stability of such a structure in this location. The only possible ways for it to be destroyed by natural means are by the yielding of the abutments to such an extent as to cause the ultimate crushing strength of the material to be exceeded, or by the direct application of such a pressure as to bring about such a stress. Mr. Williams.

Design.—One of the chief criticisms directed against arch dams has been that, by reason of the rigidity of the base, the arch action could not be developed in their lower part, and, while the speaker is not one of those who would argue that a barrel is weaker against external pressure by reason of having the heads in it, yet, to overcome this objection, and avoid as far as possible stresses of opposite signs acting at right angles to each other, a condition which certainly weakens the material's ultimate capacity to resist either one, recourse was had to a design similar to that introduced in an egg-ended boiler, and the base, as shown by Fig. 30, was made of the form of a portion of a torus or ring. The whole structure was to be 90 ft. in height, and the radius of the vertical curve of the base was 20 ft., selected so that the upward thrust at Elevation 185 would never exceed the downward pressure transmitted through the material above. By this construction it became possible to utilize the bed of the stream as an abutment at *t s*, and still permit of elastic deformations and true arch or dome action near or at the base. By inclining the radius at the junction of the torus with the superimposed cylinder at *b r*, an up-stream thrust at this point was obtained from the former tending to oppose the pressure of the water and decrease the horizontal circumferential thrust in the cylinder. Similarly, the inclination of that portion of the structure above *b r* also introduces a thrust up stream, acting likewise to decrease the horizontal circumferential thrust. Above this plane, to Elevation 250.10, the section is made up of a series of frustums of conical shells. From 250.10 to 254.25 it is a segment of a ring, and from 254.25 to the crest at 260.0 it is a segment of a conical dome. The radii of the extrados or up-stream face are shown on the left of the section, and of the intrados on the right. The maximum radius of the extrados was 67.75 ft. and that of the crest 50 ft.

Mr. Williams. The axes of the two faces are not coincident, that of the down-stream face being 2.25 ft. up stream from that of the up-stream face, thus making the dam somewhat thicker at the abutments than at the center.

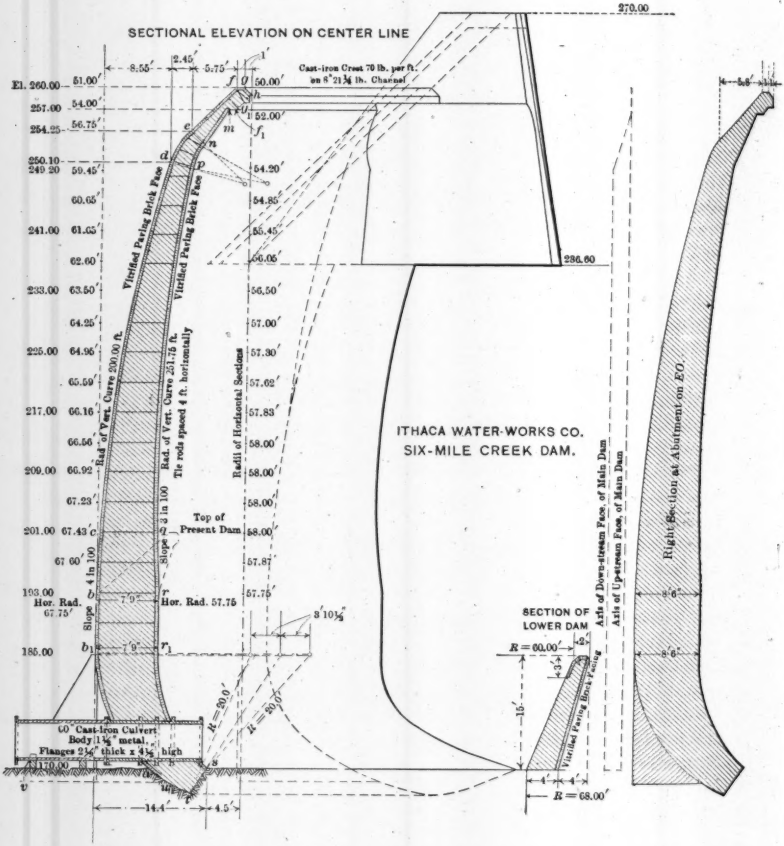


FIG. 30.

The shape of the crest was selected for the following reasons:

First, a form was desired which would discharge a maximum quantity of water at heads above 2 ft., and the one selected has been found by experiment to approximate closely to such a condition.

FIG. 1.—PREPARING THE FOUNDATIONS FOR THE SIX-MILE CREEK DAM.

FIG. 2.—UP-STREAM VIEW OF EAST END OF SIX-MILE CREEK DAM.

Second, a form was desired which would readily permit of ice climbing it, and the slope of 45° adopted answers this requirement well. Mr. Williams.

Third, a form was desired which would insure positive, certain and continuous aeration of the region behind the sheet, and the prevention of the formation of even a minute vacuum there, and the large space between the face of the dam and the falling water, in free communication with the air outside, effectually precludes the occurrence of a condition which, the speaker believes, has been, to no small extent, responsible for the failure of overfall dams in the past.

Fourth, a form was desired which would deliver the overfalling sheet well away from the toe of the section, and an inspection of Fig. 30 shows that this condition has been met.

As a further protection to the bottom, and also to insure a uniform upward thrust at $b\ r$, whether the pond were full or in flood, a second dam, 15 ft. high, was to be constructed about the middle of the gorge, 170 ft. down stream from the main dam, the overfall from which would be received in a pool formed by the old low dam already mentioned, which is 210 ft. farther down stream. This lower or middle dam was to be a segment of a frustum of a cone with a crest radius of 60 ft.

Computation of Stresses.—For preliminary purposes, the well-known formula, $T = p\ R$, wherein $T =$ the thrust or pull in the sheet, $p =$ the normal force, and $R =$ the radius of the face to which the force is applied, may be used, and, were the section cylindrical, p would be the water pressure and R the horizontal radius, and this formula would be rigidly applicable for the determination of the arch stresses. As, in the present design, the faces are generally inclined, this fact must be taken into account, and the formula becomes $T = p\ R$ sec. i, R being still the horizontal radius and i the angle of inclination of the face from the vertical. If the thickness be represented by F, then the unit thrust, $t = \frac{p\ R \text{ sec. } i}{F}$, for a section one unit high, omitting the effect of the inclination in producing a radial thrust opposite to T. As this counter thrust actually reduces T, it is evident that stresses computed by the foregoing formula will be greater than those really existing in the horizontal circumferential direction. For a flood 10 ft. in depth above the crest of the dam, which requires a run-off of 353 cu. ft. per sq. mile per sec., while the largest flood on record in this stream gave less than 100 cu. ft. and would require about 4 ft. head, the approximate thrusts in the horizontal arches by the above formula are as given in Table 9.

The thrusts in the torus base, being largely absorbed by the vertical arch of 240 in. radius, give much lower unit stresses.

For a final and more accurate determination of the stresses, the method used was as follows:

Mr. Williams.

TABLE 9.—APPROXIMATE STRESSES (IN EXCESS OF REAL STRESSES, EXCEPT ON OVERHANG NEAR CREST.)

Elevation.	Pressure of water, in pounds per square inch. p.	Horizontal radius, in inches. R.	$p\,R$ sec. i = thrust, in pounds. T.	Thickness, in inches. F.	Unit thrust, in pounds per square inch. t.
250	8.65	711	6 370	36	177
241	12.60	740	9 700	47.4	208
233	16.00	762	12 650	57	223
225	19.50	778	15 795	64.7	244
217	23.00	794	18 950	73	260
209	26.45	808	22 100	80	276
201	29.90	809	24 200	86	281
193	32.40	813	26 350	93	284

A vertical slice of the dam at the center, 1 in. thick at the up-stream face, was cut out by vertical radial planes and divided by planes normal to the up-stream face into 31 blocks, of which Nos. 1 to 7, inclusive, are on the overhang, 8 to 22 on the curved upper body, 23 is the cylinder, and 24 to 31 are on the torus base.

Beginning at the top, the force due to water pressure and that due to the weight of the block above the plane of its base are combined by a simple triangle of forces, Fig. 31, and the resultant, P, resolved into a horizontal component, P_h, and one normal to the base, P_n. For the next section, this resultant, P, is combined with the weight of the added block and the force due to pressure upon it, and a new resultant obtained which is resolved as above. Then, by Fig. 32, the forces acting on the block in question are:

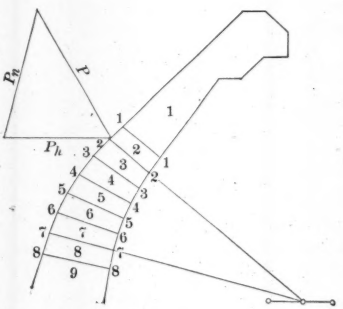


FIG. 31.

P_w = the water pressure on the face of the block acting normally thereto;

P_n = the component of the total pressure, P, normal to the base;

P_{n-1} = the component of the total pressure, P, normal to the plane of the top of the block, which = P_n for the section next above;

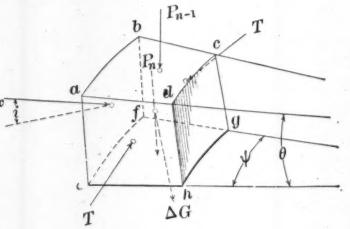


FIG. 32.

Mr. Williams.

ΔG = the weight of the block;
T = the horizontal thrust in the arch ring;
R = the horizontal radius of the up-stream face;
θ = the angle which the top and bottom faces make with each other;
ψ = the angle which the side faces make with each other;
i = the angle which the normal to the up-stream face makes with the horizon.

Strictly, the slice should have been cut out between meridional planes, in which case its thickness, if 1 in. at the crest, would have been $\frac{67.75}{50.00} = 1.355$ in. at the cylinder; or, being 1 in. thick at the cylinder, it would have been 0.738 in. at the crest. The effect of this correction would be to reduce slightly the components of G, but this is compensated for by taking a low value for the weight of the material, 140 lb. per cu. ft., and by neglecting to consider the weight of the metal in the structure.* At the base of Section 7 the theoretical thickness for a slice 1 in. thick at the cylinder would be 0.874 in., and the thinness of the sections in a radial direction at the top makes the error possibly introduced of small practical moment.

For equilibrium, by Fig. 32:

$$p_w - \left\{ (P_n + P_{n-1}) \sin. \frac{\theta}{2} - \Delta G \sin. i + 2 T \sin. \frac{\psi}{2} \cos. i \right\} = 0,$$

or,

$$2 T \sin. \frac{\psi}{2} \cos. i = p_w + \Delta G \sin. i - (P_n + P_{n-1}) \sin. \frac{\theta}{2}$$

If H = the total horizontal force carried by the horizontal arch;
F_v = the area of the vertical faces;
F_h = the area of the normal faces;
t = the unit pressure, per square inch, on the horizontal arch at the center of the section;
s = the unit pressure, per square inch, on the vertical arch at the base of the section;

then

$$H = 2 T \sin. \frac{\psi}{2} = \left[p_w - (P_n + P_{n-1}) \sin. \frac{\theta}{2} \right] \sec. i + \Delta G \tan. i. \quad (3)$$

But, by the dimensions of the block, $\sin. \frac{\psi}{2} = \frac{1}{2}$ in. $\div R$, in inches; therefore,

$$H = 2 T \frac{\frac{1}{2}}{R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

or

$$T = H R \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

* The weight of the concrete alone, without the added boulders, was 141.4 lb. per cu. ft. The brick facing weighed 144.4 lb. per cu. ft. and the iron and steel averaged more than 0.8 lb. per cu. ft. of the entire mass, one-half of this weight being within 2 ft. of the crest.

Mr. Williams.

TABLE 10.—Analysis of Stresses in Six-Mile Creek Dam, (Slide-Rule

Section.		Elevation: Feet above Cayuga Lake = City Datum.		Total Pressure above Base of Section. P. Pounds.		Total Pressure Normal to Base of Section. P_n. Pounds.		Water Pressure Normal to Face of Section. p_w. Pounds.		Weight of section. ΔG. Pounds.
No.	Location.	Base.	Center.	Full pond.	10-ft. flood.	Full pond.	10-ft. flood.	Full pond.	10-ft. flood.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1		.....	256.7	234	588	292	686	94.8	467	157.0
2		254.00	254.0	275	698	340	775	118.6	535	178.0
3		253.25	253.5	327	802	378	800	30.2	81	22.5
4		252.75	253.0	374	874	403	820	35.4	87.4	24.0
5		252.05	252.5	432	978	440	880	38.5	89	25.4
6		251.15	251.6	486	1 084	471	940	44.2	96	26.6
7		250.25	550.7	547	1 180	503	985	47.8	99	29.0
8	On overhang.	249.30	249.8	608	1 216	545	965	51.0	102	30.0
9		245.0	247.1	974	1 800	808	1 300	286.0	525	162.0
10		241.0	243.0	1 396	2 400	1 075	1 605	364.0	578	172.5
11		237.0	239.0	1 920	3 120	1 382	1 975	449.0	663	191.5
12		233.0	235.0	2 520	3 920	1 690	2 330	533.0	746	208.0
13		229.0	231.0	3 200	4 810	2 030	2 690	616.0	827	222.0
14		225.0	227.0	3 963	5 770	2 383	3 070	698.0	910	236.0
15		221.0	223.0	4 813	6 830	2 765	3 500	777.0	989	248.0
16		217.0	219.0	5 725	7 932	3 135	3 900	859.0	1 065	261.5
17		213 0	215.0	6 710	9 110	3 485	4 300	940.0	1 150	270.5
18	On curved upper body.	209.0	211.0	7 770	10 400	3 875	4 710	1 020.0	1 229	281.0
19		205.0	207.0	8 925	11 730	4 240	5 050	1 102.0	1 310	296.0
20		201.0	203.0	10 130	13 150	4 620	5 420	1 187.0	1 393	313.5
21		197.0	199.0	11 440	14 690	5 000	5 880	1 269.0	1 478	324.0
22		193.0	195.0	12 855	16 260	5 400	6 220	1 352.0	1 560	326.0
23	Cylinder.	185.0	189.0	15 825	19 490	6 050	6 900	2 950.0	3 202†	682.0
24		182.8	183.9	16 620	20 480	6 200	7 050	833.0*	884†	158.0
25		180.7	181.8	17 400	21 300	6 300	7 160	844.0*	894†	158.0
26		178.6	179.7	18 140	22 120	6 380	7 230	856.0*	907†	155.7
27		176.6	177.4	18 880	22 890	6 350	7 260	866.0*	918†	153.6
28		174.8	175.7	19 530	23 540	6 330	7 230	875.0*	925†	150.0
29		173.1	174.0	20 120	24 200	6 200	7 160	885.0*	933†	142.5
30		171.5	172.3	20 600	24 800	5 950	7 020	888.0*	938†	134.1
31	On torus base.	170.0	170.7	21 050	23 350	5 680	6 830	893.0*	942†	124.6
										Σ 5 863.5

* Component of water pressure on down-stream face subtracted.
§ Strictly, K, for center of gravity, should be used, since forces are combined there. By so doing the horizontal thrusts would be reduced about 2% at the top and 7½% at the cylinder.

OMITTING INFLUENCE OF ATTACHMENTS AT SIDES AND BOTTOM. Computations.)

Sine $\frac{\theta}{2}$	Sec. i.	Tan. i.	Radius of extrados of horizontal arch. R. Feet.	Areas. Vertical face of section. F_v. Square feet.	Areas. Base of section. F_b. Square inches.	Total Force Carried by Horizontal Arch. H. Pounds. Full pond.	Total Force Carried by Horizontal Arch. H. Pounds. 10-ft. flood.	Thrusts in Arches, in Pounds per Square Inch. Horizontal. $t = \frac{H R}{12 F_v}$ ‖ Full pond.	Horizontal. 10-ft. flood.	Vertical. $s = \frac{P_n}{F_b}$ ¶ Full pond.	Vertical. 10-ft. flood.
(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)
0.000	1.414	1.000	54.40	13.45	26.2	291.0	817	98	275	11.1	26.2
0.000	1.414	1.000	54.70	15.26	26.9	345.0	934	103	279	12.8	28.8
0.487	1.345	0.896	56.95	1.93	28.2	36.7	76	90	187	13.4	28.4
0.487	1.230	0.713	57.60	2.06	29.8	14.5	28	33.7	65	13.5	27.5
0.487	1.161	0.600	58.15	2.18	31.1	0.7	23	1.5	51	14.1	32.0
0.487	1.108	0.414	58.60	2.28	32.1	10.9	20	23.3	43	14.7	33.3
0.487	1.068	0.306	59.00	2.48	32.8	9.3	15	18.4	30	15.3	30.0
0.010	1.046	0.277	59.30	2.57	33.2	30.3	54	58.3	104	16.4	29.1
0.010	1.040	0.270	60.05	14.20	40.7	382.7	566	117	199	19.9	32.0
0.010	1.035	0.250	61.15	15.25	45.9	379.2	611	127	204	23.4	35.0
0.010	1.030	0.237	62.12	16.98	50.1	482.4	691	147	211	27.6	39.4
0.010	1.027	0.228	63.05	18.55	55.3	583.5	770	160	218	30.6	42.1
0.010	1.022	0.200	63.87	19.85	57.8	636.5	839	170	225	35.1	46.5
0.010	1.017	0.171	64.60	21.10	62.3	705.3	908	180	231	38.3	49.3
0.010	1.014	0.150	65.32	22.45	65.7	774.9	972	188	237	42.0	53.3
0.010	1.011	0.130	65.87	22.90	69.2	841.0	1 086	202	248	45.3	56.4
0.010	1.007	0.110	66.36	24.68	72.3	909.8	1 106	204	248	48.2	59.5
0.010	1.005	0.089	66.74	25.40	76.3	975.0	1 171	213	257	50.8	61.7
0.010	1.003	0.065	67.08	26.70	79.6	1 043.3	1 234	218	258	53.3	63.5
0.010	1.002	0.044	67.33	28.36	81.6	1 113.8	1 304	220	258	56.7	66.8
0.010	1.001	0.044	67.52	29.50	85.0	1 188.3	1 380	226	263	58.8	68.6
0.010	1.001	0.044	67.67	29.50	87.6	1 264.3	1 455	241	278	61.5	71.0
0.0022	1.000	0.000	67.75	62.00	87.6	2 883.0	3 125	262	285	69.0	78.8
0.053	1.0016	0.0574	67.70	13.60	87.0	482.9	487	200	202	71.5	81.0
0.053	1.0135	0.1648	67.46	13.60	85.5	157.0	117	65	48.3	73.7	93.7
0.053	1.0374	0.2730	67.00	13.30	84.0	148.0	107	62	44.9	75.4	86.2
0.053	1.075	0.3940	66.28	13.10	81.6	164.0	116	69	48.9	77.6	89.0
0.053	1.128	0.5223	65.43	12.80	78.5	157.0	99	67	42.1	79.6	92.2
0.053	1.201	0.6657	64.32	12.10	74.3	175.0	110	78	48.8	83.5	96.3
0.053	1.300	0.8309	63.07	11.50	69.9	205.0	131	94	59.8	85.1	100.4
0.053	1.434	1.0282	61.60	10.70	64.4	259.0	160	124	75.7	88.3	106.0

†Component of water pressure on down-stream face, due to 8 ft. head on lower dam subtracted.

‖ At center of sections.

¶ At base of sections.

Mr. Williams.

and $$t = \frac{T}{F} = \frac{H\,R}{F} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

and $$s = \frac{P_n}{F_b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

For R and F, in feet, Equation 6 becomes

$$t = \frac{H\,R}{12\,F} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

Table 10 presents the elements of this computation for a full pond and for a 10-ft. flood.

Comparing the stresses induced for the two cases, Columns 20 to 23, the interesting fact is discovered that in this structure the unit stresses in the horizontal arches of the torus base—the weakest spot, apparently, if the design be judged by inspection simply—are less for the case of a flood than for that of a full pond, and, in spite of the apparently thin section at the toe, the maximum stress is only 124 lb. per sq. in., while the maximum anywhere in the structure, under assumed conditions far beyond any possible contingency, is less than 285 lb. per sq. in. Using the radius of the center of gravity of the cylindrical block, which is approximately 63 ft., the maximum unit stress in the dam is seen to be $\frac{63.00}{67.75} \times 285 = 265$ lb. per sq. in., or 19.08 tons per sq. ft.

Owing to the thicker section of the dam as it approaches the abutments, the corresponding maximum pressures on the rock are: for the east abutment, 247 lb. per sq. in., or 17.8 tons per sq. ft.; and, for the west abutment, 211 lb. per sq. in., or 15.2 tons per sq. ft. By way of comparison, it may be recalled that the pressures on the foundations of the Rookery Building, in Chicago, and those of the old Brooklyn Bridge are given as 400 lb. per sq. in., or 28.8 tons per sq. ft., while the concrete and low-grade rubble base of the Washington Monument is subjected to loads of 525 lb. per sq. in., or 37.8 tons per sq. ft. There are also a number of unreinforced concrete-arch bridges abroad which have been standing several years, in which stresses greater than 300 lb. per sq. in. either exist continuously or occur frequently, aside from those due to temperature changes and rib shortening.

The stresses near the crest are amply provided for by the crest casting and the steel channels in that portion of the structure.

Having now considered the conditions of full pond and flood, it remains to enquire as to the stresses in certain parts of the structure at low water, and when the pond is empty, should the latter condition ever occur after the completion of the work. Examining the horizontal thrusts due to the weight of the dam, it was found that for Sections 5 to 8, inclusive, the outward thrusts are decreasing downward, whence, in an ordinary dome, tension would occur in this region, and should in that case be resisted. Any yielding in the haunches,

Mr. Williams.

however, must be accompanied by a lowering of the crest at the center, and, because of the rigidity of the abutments in this case, which prevents spreading along the chord of the dam, any lowering of the crest will be resisted by the hyperbolic arch formed along the vertical plane through the crest, and, consequently, the hoop usually supplied to a dome at the so-called joint of rupture is not needed here, although, to relieve the small tensions which might occur while the vertical arch resistance was developing, a hoop of 4 by $\frac{3}{4}$-in. steel was provided at the haunch.

At the top of the torus base, similarly, tension would occur with the pond empty, were it not that a system of piers introduced under the heel of the dam acts as a support for the upper masonry at such times. These piers have no bond with the body of the dam, which is free to move away from them when loaded, but they act simply as wedges to keep the structure erect when there is no pressure on the back, and prevent tensile stresses at the top of the torus base.

As already stated, in the design of the Six-Mile Creek Dam, an attempt was made to eliminate, as far as possible, the influence of the beam or cantilever action which plays so extensive a part in the stresses of such curved dams as the one described by the authors, and the Zola and Sweetwater Dams, and even the more rationally designed Bear Valley Dam. To discover how successfully this has been accomplished, the midsection of the dam under a 10-ft. flood was subjected to an analysis similar to that presented for the Lake Cheesman Dam. A vertical slice of the dam, 1 ft. in thickness circumferentially, was taken, and, to simplify computations, rectified by projection upon a vertical tangent to the cylindrical portion, and this was divided into six sections 15 ft. in height. The average moment of inertia and the resulting deflections for each section were computed as for a beam, and then the average deflection of each section as an arch. The results are presented in Table 11, wherein, adopting the authors' notation:

a = height of section = 15 ft.;
D = the deflection;
I = the average moment of inertia for the section;
E = the modulus of elasticity;
X = the horizontal component of load on the section;
T = the thrust of the horizontal arch, the average value for a layer 1 ft. thick being used;
L = length of the arc of the arch;
A = area of arch layer 1 ft. thick;
α = one-quarter arc of arch before loading.

From Columns 18 and 19, the loads carried by the two systems of forces being inversely as the deflections under the same load, it is seen that at no point above Elevation 185, which is only 15 ft. above

Mr. Williams.

TABLE 11.—COMPARISON OF ARCH AND BEAM STRESSES IN THE SIX-MILE CREEK DAM.

	For Beam.								For Arch.							Percentage of Load Carried by:	
	Elevations.		Thickness, in feet.														
	Section.	Top.	Bottom.	Top.	Bottom.	Load, X, in pounds.	Moment of inertia, I.	$\frac{a^3}{6EI}$	ED for beam.	Elevation.	Radius, R, in feet.	Area of Ring 1 ft. thick, A, in square feet.	α	$L = \pi R \frac{4\alpha}{180°}$, in feet.	T, in pounds per square foot.	$ED = \frac{TL \cot. \alpha}{2A}$.	$\frac{ED \text{ for Beam}}{ED \text{ for Arch}} = \frac{D \text{ for Beam.}}{D \text{ for Arch.}}$	Arch.	Beam.
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	290	245	1.0	3.45	16 400	1.20	408	888 320 000	252.5	55.93	2.22	26° 48′	104.65	27 200	1 270 000	695	99.85	0.15
2	245	230	3.45	4.93	30 400	6.32	$\frac{E}{89}$	684 800 000	237.5	60.33	4.18	24° 07′	101.61	30 200	882 000	808	99.91	0.09
3	230	215	4.93	6.19	44 400	14.51	$\frac{E}{38.8}$	469 950 000	222.5	62.59	5.56	22° 59′	100.42	33 850	721 000	652	99.85	0.15
4	215	200	6.19	7.25	58 400	25.35	$\frac{h^3}{22.2}$	298 170 000	207.5	63.61	6.72	22° 31′	100.00	37 100	666 000	448	99.77	0.23
5	200	185	7.25	7.75	68 300	35.20	$\frac{E}{16}$	154 900 000	192.5	63.81	7.50	22° 25′	99.86	40 100	647 000	239	99.54	0.46
6	185	170	7.75	5.70	71 100	25.93	$\frac{E}{21.7}$	46 505 000	177.5	63.83	6.72	22° 58′	100.45	10 380	188 000	254	99.61	0.39

the base, does the beam carry one-half of 1% of the load, and it is also apparent that at half this distance above the base the beam cannot carry as much as 1 per cent. Mr. Williams.

Therefore, it may be concluded that the purpose of the designer, in this respect, has been accomplished, and that the stresses presented in Table 10 represent fairly the conditions in the structure.

At the west abutment, where, as already stated, the rock did not rise to the height of the crest, the thrust in the upper portion of the dam was taken up by a concrete abutment rising to Elevation 270, beyond which the dam was continued into the hillside as a series of elliptical half cylinders, the section of which at 45° was a circle, and which were inclined at 45° and cut off 10 ft. above the crest of the dam by a horizontal plane, the cylinders being supported by wedge-shaped piers under their springings.

As this part of the design has no bearing on the case of the Cheesman Dam, it will not be discussed further at this time.

Material and Construction.—The questions of material and construction have no particular bearing upon the Cheesman Dam, wherein the material was unquestionably the most desirable and best that could have been used, but, as considerable interest has been manifested in these matters, and as a description of the Six-Mile Creek Dam would be incomplete without them, they will be added here.

The body of the Six-Mile Creek Dam is of concrete composed of 1 part Alsen's imported Portland cement, 2 parts creek sand, 2 parts creek gravel and 2 parts broken stone from drift boulders, crushed to pass a 4-in. ring or less.

The voids in the sand amounted to about 42% of its volume. Mortar briquettes, 2 of sand to 1 of cement, 7 days old, indicated that it had a strength in tension equal to about two-thirds of that of standard sand.

The creek gravel was ordinary drift mixed with fragments of the shale rock of the region. Where the latter appeared as flat stones they were broken up or raked out.

The crushed stone contained about 15% of selected shale from the excavation in the rock walls, the remainder being field boulders. Flat stones were rejected both before and after crushing, as far as a reasonably close inspection discovered them.

The faces of the dam were of a single course of vitrified paving brick laid in a mortar of 1 part Alsen's cement, 1 part creek sand and 1 part crusher dust, and were anchored into the body by bent steel anchors, $\frac{1}{2}$ by $\frac{1}{8}$ by 7-in., turned up $\frac{1}{2}$ in. at each end, placed at every fifth brick in every fifth course. On the up-stream face of the torus base the bricks were laid with the flat exposed, elsewhere with the edge exposed.

The brick used was that known as Catskill block, 3 by 4 by 9 in., a

Mr. Williams. very thoroughly vitrified shale brick, weighing 144.4 lb. per cu. ft. They were generally burned so highly as to be distorted considerably by the heat. Four samples, immersed in pails of water for four months, increased in weight less than one-tenth of 1%, and, when tested endwise in compression, failed by splitting lengthwise with a sharp report, at pressures varying from 2 300 to 4 600 lb. per sq. in.

Next inside the brick is a 3-in. mortar face of the same mixture as that used for the joints in the brickwork, which was laid at the same time as the concrete body, being separated therefrom by a plate of iron, until both were placed, when the iron was withdrawn and all were rammed together. Within this mortar face, and as close to the brick as convenient, *i. e.*, about 1 in. away, were set, above Elevation 185, bands of 3 by $\frac{3}{16}$-in. steel extending around the structure every 4 ft. in height, and united through the dam every 4 ft. horizontally by steel rods, $\frac{5}{8}$ in. in diameter, with a nut at each side of the bands. At Elevation 185, on the up-stream side, a band of 4 by $\frac{3}{4}$-in. steel was used and connected to the opposite 3 by $\frac{3}{16}$-in. band in a similar manner, to provide for possible tensions from pier to pier when the pond was empty. Over this steel skeleton, which was held in place by the horizontal rods extending into the brick faces, there was laid or hung a netting of crimped $\frac{3}{16}$-in. longitudinal and $\frac{1}{8}$-in. vertical wire of 4-in. mesh, extending from abutment to abutment on each face, and lapped one mesh and wired together at the horizontal joints of the sheets. All iron and steel was grouted carefully by dipping it in a trough as soon as it came on the work and before it had time to rust, and the bands and netting were placed as close to the outer faces as possible, their purpose being to distribute the stresses due to temperature changes and thereby prevent local cracks. The mortar and concrete were mixed in a Ransome mixer located about 150 ft. up stream from the dam, and the material was placed very wet. Into the body of the concrete were forced one-man stones as each layer was put in, they being carefully set with bed planes normal to the line of thrust, and were left projecting about half their height when a section was completed.

The brick walls were first laid up to a height of about 4 ft., No. 10 steel wires being bedded in every fifth course on the up-stream side, and, after setting about two days, the concrete was placed between them, they making the forms after Elevation 185 was reached. No deformation of the walls was detected in any part of the work, although braces on the up-stream side, used at first, were dispensed with entirely as the work progressed, and the down-stream side was left entirely unsupported all the time. Fig. 2, Plate IX, shows clearly the method of construction and the appearance of the work.

The foundations were carried down to sound rock, usually from 5 to 6 ft., but in one case, for a short distance, to 18 ft., below the bed

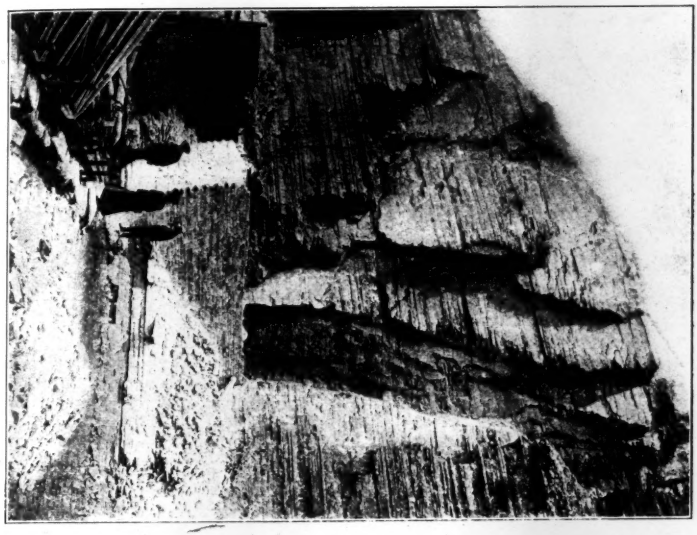


FIG. 1.—SECTION OF BASE OF SIX-MILE CREEK DAM.

FIG. 2.—THE SIX-MILE CREEK DAM.

of the stream. The longitudinal seams in the bottom were followed by 2 or 3-in. drill holes for from 4 to 6 ft. below the bottom of the foundations, and the holes filled with plastic clay well rammed. Fig. 1, Plate X, shows the end sections of the torus base completed, and excavation in progress for the middle. Mr. Williams.

Handling of the Stream.—As it was impossible to divert the stream from the gorge, it was necessary to provide for carrying it through the work, and the design contemplated the erection of the portions of the torus base at the abutments and in front of the piers, leaving the intermediate spaces open, but making all excavations, the stream being diverted from side to side during the work. Arches were to be sprung across the openings thus left, and the dam completed above Elevation 185, leaving passages through the base of sufficient capacity to deliver an ordinary flood. A permanent cast-iron culvert, 5 ft. in diameter, was also provided through the base and controlled by a gate. After the upper portions of the dam were completed, the openings through the base were to be filled, one at a time, at low water, the culvert then being able to carry the flow. Fig. 1, Plate XI, looking up stream, shows the base at the west end of the dam, and the low-water flow of the creek passing through the opening there while the center of the base is building.

The Dam as Built.—When it became noised abroad that a dam 90 ft. high and but 8 ft. thick at the base was to be built only two miles above Ithaca, to form a lake of 60 acres area, many people immediately saw visions of a Johnstown flood, and protests began to appear in the public prints. The plans, meanwhile, had been referred to four prominent members of this Society, the first of whom withdrew without making any report either favorable or unfavorable, and the other three, fully cognizant of all the conditions, including the action of the first, reported an unqualified approval. In due time bids were called for, and six bidders submitted proposals. Of these bidders, four were experienced engineers and contractors, three being members of this Society. Not one of them, after examination of the plans, specifications and location, expressed any doubt as to the stability of the structure. The tenders were received on unit prices for the several kinds of work involved, and the gross bids, exclusive of cement, which was furnished by the Water Company, based on the Engineer's estimates of quantities, were as follows: $63 365; $55 795; $44 280; $38 957; $35 360, and $34 488.

After work was begun, discussion of the structure continued, and a few so-called engineers, who had never seen the plans, expressed themselves in condemnation of the structure. Several others, after examining plans and location, expressed approval, and others still, perhaps more discreet than either, said nothing. The citizens of Ithaca invoked the aid of the State Engineer, the State Flood Com-

TABLE 12.—DISTRIBUTION OF STRESSES AND ARCH THRUSTS IN BRICK-FACING ARCH.

Height 52 in., Thickness 4 in., Span 90 ft., Radius of Extrados 50.08 ft.

Position.	Deflections of Arch Crown.		Water pressure, in pounds per square inch.	Part of Total Load Carried by Arch and Arch Thrusts.									
				At crown.		At $\frac{1}{10}$ span.*		At $\frac{2}{10}$ span.		At $\frac{3}{10}$ span.		At $\frac{4}{10}$ span.	
	Arch carrying entire load.	Vertical beam carrying entire load.		Percentage of load.	Thrust, in pounds per square inch.	Percentage of load.	Thrust, in pounds per square inch.	Percentage of load.	Thrust, in pounds per square inch.	Percentage of load.	Thrust, in pounds per square inch.	Percentage of load.	Thrust, in pounds per square inch.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
At top	$\frac{78\,000}{E}$	$\frac{135\,500}{E}$	0.346	63.5	88	68.5	41	74.3	45	81.3	49	89.7	54
13 in. below top	$\frac{185\,000}{E}$	$\frac{94\,200}{E}$	0.82	33.8	48	38.9	55	45.8	65	56.2	80	71.9	102
At mid-height	$\frac{291\,000}{E}$	$\frac{51\,800}{E}$	1.28	15.12	34	18.18	41	22.9	51	30.8	69	47.2	106
13 in. above bottom	$\frac{398\,500}{E}$	$\frac{15\,950}{E}$	1.75	3.84	12	4.76	14	6.25	19	9.09	28	16.67	51
At bottom	$\frac{504\,000}{E}$	0	2.22	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0

* Fractional spans measured from abutment.

Mr. Williams.

Mr. Williams.

mission, and, finally, the State Health Commissioner, and injunction proceedings were threatened, but, apparently for lack of the necessary kind of engineering advice, were never brought. The breaking out of a typhoid epidemic, due to the water supply, but most probably not, as charged, to the infection of the water by work on this structure, raised public excitement to such a pitch that an almost unanimous vote in favor of municipal ownership was taken, although a similar proposition had been defeated less than a year before; and the State Health Commissioner announced at a mass meeting that his consulting engineers had declined to approve the design of the dam; but, he afterward stated, over his signature, that no official report had been made to him on the subject.

In consequence of all this, the company decided to stop the construction of the dam at a height of 30 ft., and, accordingly, it was built on the original lines to Elevation 193 and finished at Elevation 201, with a crest 60 ft. in radius overhanging on the down-stream side and having a 45° up-stream slope, as shown in broken lines in Fig. 30, and photographically in Fig. 2, Plate X, and Fig. 2, Plate XI, the former being a view of the up-stream face of the dam at the east end, and the latter the completed dam as it appears from the west abutment.

A flood of 3 ft. depth above the crest went over the dam on August 31st, 1903, before the runway used in its construction had been removed. By descending a ladder at each end of the dam, it was possible to look under the sheet and observe the face of the dam. A single leak, supplying a jet about half the size of a lead pencil, appeared about 8 ft. above the base. At this time the up-stream face of the dam, at the base, where the brick was laid up against forms, had not been pointed. Since pointing, neither this nor any other visible leaks have appeared.

A Test of the Brick Facing Arch.—During the construction, when the concrete was completed to Elevation 185 and the brick walls were up about 4 ft. higher, ready to receive the concrete filling, the work being in a similar condition to that shown by Fig. 2, Plate IX, the brick arch was subjected to an interesting test.

The brick walls, 4 in. thick, had been carried to a height of 52 in. above the top of the concrete on June 19th, 1903. The down-stream wall had all been laid within ten days, having been finished less than a week, and the up-stream wall laid within five days, a stretch of about 6 ft. near the east end, where the runway had been located, having been finished that afternoon. A heavy rain occurred the next day, and the water rose to such a height during the night as to overflow the dam to a depth of 0.8 ft. It began going over about 3 A. M., and was still overflowing at 9:30 A. M., the maximum height probably lasting for 2 or 3 hours.

Mr. Williams.

As the water rose it apparently dissolved the mortar in the short portion of the wall, about 6 ft. long, laid the day before, and broke through there before it went over the top, for the down-stream wall opposite this place was bulged, for about 10 ft., from nothing to about 1½ in., the maximum being at about two-thirds of its height; and the up-stream brick wall, near the opening thus formed, was tipped over up stream, falling to the bottom of the pond above the dam, thus indicating that a powerful stream of water had recoiled from the lower wall against the concave side of the then unsupported upper wall, and wrecked it. At the far end of the dam, the up-stream wall fell over upon the concrete of the base, showing that the recoil had spent itself before reaching that part of the work.

In spite of having received what must have been a very severe shock, this brick arch, 52 in. high, 4 in. thick, and of 90 ft. clear span, with an up-stream radius of 58 ft. 1 in., stood throughout the flood, and, when the water subsided to a level with the top, no leaks appeared through the wall, except where openings had been left for the insertion of tie rods, and the wall was used, as originally intended, without any repairs or alterations.

TABLE 13.—BEAM STRESSES IN BRICK-FACING ARCH.

Position from abutment.	PART OF TOTAL PRESSURE CARRIED BY BEAM 1 IN. WIDE.		Distance of center of pressure above base, in inches.	Moment at base, in inch-pounds.	MAXIMUM STRESSES ON HORIZONTAL MORTAR JOINTS, IN POUNDS PER SQUARE INCH.		
	Percentage.	Pounds.			Tension.	Compression.	Shear.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crown.....	87.3	58.32	17.2	1 004	372	380	14.5
⅛ span....	85.2	56.89	16.9	962	357	365	14.2
¼ span....	82.1	54.81	16.5	905	335	343	13.7
⅜ span....	77.2	51.56	15.8	814	301	309	12.9
½ span....	68.0	45.45	14.2	645	238	246	11.4

The stresses set up in this thin arch have been examined by a process similar to that used by the authors, and the results are presented in Tables 12 and 13. One point, not mentioned before, is to be noted, *viz.*, that, in considering a section of a dam as a beam, its action differs from that of an ordinary horizontal beam, since it does not begin to deflect until the load becomes sufficiently great to overcome the moment of the weight of the structure itself about some point in its base; and, if upon a yielding foundation, this point will not be the toe, as is usually incorrectly assumed in dealing with the overturning of gravity sections. Allowance for the weight of the

material has been made in the case of this brick arch, but was omitted in the analysis of the Six-Mile Creek Dam, and, apparently, in that of the Lake Cheesman Dam. In the former, by reason of its peculiar profile, it appears that the moment of the weight might become a considerable factor, and while not affecting greatly the deflections in the upper portions, might, to an appreciable degree, reduce that at the bottom, and hence the total deflections of the structure as a beam. Enquiring into this matter, the total weight of a slice of the dam 1 ft. wide is found to be, by Column 11, Table 10, $12 \times 5863.5 = 70\,362$ lb.; and, by inspection of the profile, it appears that the center of gravity will be somewhere within 5 ft. of the point of rotation at the base. The moment of the weight of the structure against overturning, therefore, is less than 350 000 ft.-lb. The total water pressure on the 1-ft. slice of the dam under a 10-ft. flood is $50 \times 62.4 \times 90 = 280\,800$ lb., and the point of application of its resultant is approximately 32.5 ft. above the base. The overturning moment, therefore, is $32.5 \times 280\,800 = 9\,126\,000$ ft-lb. The introduction of the weight factor into the computations of Table 11, consequently, would reduce the bending moment for the beam section less than $\frac{280\,800}{9\,126\,000} = 3.07\,\%$, an amount too small to affect to an important degree the results there set forth. Mr. Williams.

The values of the stresses given for the brick arch in Table 13 are computed on the usual assumption that the modulus of elasticity of the brick and the mortar joints is the same in compression as in tension. The tensions represent, not only the tensile strength of the mortar in itself, but also its adhesion to the brick surfaces, and the same is to be said for the shears. Tests by briquettes made of 2 parts of creek sand to 1 of cement, with 17% of water, when one week old, gave 270 to 300 lb. per sq. in. tensile breaking stress, and at two weeks 375 to 400 lb. The replacement of 1 part of the sand by 1 part of the crusher screenings probably increased the strength of the mortar somewhat, so that a tensile stress of 372 lb. is within the probable strength limit of the mortar ten days old.

It is noticeable, in such a long and slender arch as is represented by the brick wall, that the arch deflections at the crown would be much greater than the deflections as a beam, and the values for the portions of the loads taken by the two systems are to be looked upon as limiting approximations. The beam cannot be expected to carry more nor the arch less than these quantities. It is noticeable, however, that the thrust of the arch increases toward the abutments by reason of its deflection becoming less. As the radius, thickness and water pressure are unchanged from point to point, this increase can only be provided for or resisted by the absorption by the arch of some of the stress credited to the beam, or by the setting up of shearing

Mr. Williams.

stresses of varying amounts, along horizontal planes. It follows that any complete analysis of the stresses in this arch, or any one in which the beam action is an important factor, must involve the consideration, not only of these, but also of secondary stresses set up by the action at right angles of primary stresses of opposite signs, all of which makes a very complicated problem.

In the Six-Mile Creek Dam no such uncertainty exists, as the beam action is practically eliminated, and the only secondary stresses to be considered are those due to rib shortening and temperature, the former of which cannot be nearly as serious as in the case of an arch bridge, with its necessary bending moments under both dead and live loads, and the latter of which is of less consequence than in such a structure, because the range of temperature is necessarily less.

It appears, then, that in the design of the Six-Mile Creek Dam there is presented a structure, which, for all practical considerations, acts wholly as an arch under uniform normal pressures, the equilibrium curve for which condition coincides with the center line of the section, and this seems to prove it to be as near an approach to the ideal as engineers are usually able to accomplish in structures of its magnitude.

The work was executed, under the speaker's personal supervision, by Messrs. Tucker and Vinton, Inc., of New York City, who used every endeavor to produce a most creditable structure. The speaker was assisted upon the construction by S. C. Hulse, Jun. Am. Soc. C. E., and Mr. Weston E. Fuller, whose care and interest in the work also merit commendation.

Mr. Harrison.

C. L. HARRISON, M. AM. SOC. C. E. (by letter).—Engineers who have attempted the solution of similar problems take a keen interest in the subject, as indicated by the number of discussions presented The description of the design and execution of other works at different locations and under different conditions, which is really additional information on the general subject, is especially instructive in that it shows such a possible wide range of design and evinces a desire to improve on what has been accomplished in the past. Such a spirit leads to real progress.

It appears that some of the members who have discussed the paper considered it a presentation of the relative merits of the arch and the straight masonry dam, while the real object of the authors was to place before the Society an account of the design and construction of a dam at this particular location. Also, it does not appear to be fully understood that the dam was designed with a gravity section and was curved in plan because this form fitted the contour of the ground. It was believed the arch would give additional strength, but the attempt to determine the amount approximately was an after-consideration.

Omissions.—To present all the features in detail would require a

very long article, and perhaps befog the main question. A description of the spillway and regulating devices was purposely omitted in the hope that A. E. Kastl, M. Am. Soc. C. E., the Chief Engineer, who completed the work, would take up these subjects in a discussion of the paper. The suggestion of Mr. Kuichling that more space should have been given to the hydrology of the catchment basin is a very worthy one, but this basin presents so many interesting features and is so markedly different from those of the Middle and Eastern States, that the subject is worthy of a separate and lengthy paper, which may be presented at some future time. However, it may now be noted that this catchment basin is wholly within the Rocky Mountains, at an elevation varying from 7 000 to 12 000 ft. above sea level. The precipitation is nearly all in the form of snow. These conditions make a flood during the winter season impossible. In fact it may be several months before the snow which falls in the deep gulches and on the northern sides of the mountains melts, and, in the meantime, a portion of it evaporates, and a considerable portion of it enters the ground, resulting in a lower and less rapid run-off than for water-sheds where the precipitation is in the form of rain. Mr. Harrison.

Alternate Designs.—By referring to Fig. 1, Plate IV, it will be seen that a curve of about 400 ft. radius is as small as could be used, and the location gives the shortest span across the cañon. The long chord or span of the dam as actually constructed is about 610 ft., measured between the ends of the center line of the top at Elevation 217 ft. The suggestion of Mr. Shirreffs, that a radius of 260 or 300 ft. might have been adopted with economy, could not possibly be followed, for the reason that the dam would not reach across the valley at that elevation. Likewise, the suggestion of Mr. Williams, that an arch dam of thin section might have been used, would probably not have been followed by him, as his suggested maximum span of 500 ft. for such a design is greatly exceeded at this location. It is probable that no engineer would build a high dam, depending wholly upon the arch for stability, with a radius greater than 300 ft. By referring to Plate IV, it will be seen that the bottom and sides of the cañon below Elevation 35 ft. (foundation of dam at approximately—10 ft.) are very irregular, and the width of the cañon is about one-fifth the base thickness of the dam. Between Elevation 35 and Elevation 90, the masonry is nearly in the shape of a keystone wedged in between the walls of the cañon. As pointed out by Mr. Wisner, who visited the work while in progress, this holds the lower 100 ft. of the masonry firmly in position and prevents any tendency to either overturn or slide. Mr. Wisner suggests that a base width of two-thirds of that used might have been adopted with safety, though the amount of masonry thus saved would not have been large. This is probably true, since the conditions make it practically impossible for the higher unit stresses

Mr. Harrison. at the toe, given in Table 3, to be realized, but the additional width of base gives a factor of safety which could be obtained at small cost. Generally, the suggestions for a different design are for a lighter section than the one used, and no doubts have been expressed as to the stability of the structure. When the consequences of failure are very great, the engineer should build abundantly strong. In addition to satisfying the conditions of stability from an engineering standpoint, the design should not be such a radical departure from established practice as to make it impossible or difficult to finance the project.

Temperature Stresses.—The phenomena of materials expanding and contracting on account of change of temperature is not of recent discovery but has long been known. In metal bridge designs, this is provided for by placing expansion joints at certain intervals. Also, in masonry retaining walls, expansion joints are often placed at intervals of 25 to 50 ft., not with the expectation of preventing temperature cracks, but to confine them to those particular lines. The only way to prevent them is to maintain a uniform temperature, which is practically impossible. As pointed out by Mr. Gould, the wider range of temperatures is confined to a few feet on the outside, and the interior of a heavy mass of masonry may undergo little or no change of temperature. This makes it possible for such cracks to be of only superficial extent, even though they may show on both sides and the top of the structure. In view of the fact that so few actual determinations have been made of the changes of temperatures in the interior of large masses of masonry, it may be well to postpone a discussion of the subject until such data are available. It is to be hoped that the information obtained at the Boonton Dam, referred to by Mr. Smith, will be fully presented to the Society as soon as the observations are completed. As far as known, there has been no failure of dams due to temperature cracks, and engineers who have had to deal with them apprehend but little danger from their existence. Mr. Gowen infers from his observations on other dams that the arch form is less likely to develop temperature cracks than the straight dam. This is probably true to a greater degree for a thin arch than for a thick one. It is not surprising that one small temperature crack of a maximum width of $\frac{1}{32}$ in. developed in the Cheesman Dam, but it is surprising that only one developed when the extent to which they have shown in other structures is considered. As stated by Mr. Wisner, this crack closed during the following summer.

The maps and plans mentioned by Mr. Maltby were studied with interest, but neither was adopted in carrying out the work. The locations of the dam and spillway were fixed by the contour of the ground and the masonry left intact after the destruction of the rock-fill dam. All the maps used in the paper were made from original surveys, and it is not surprising that two contour maps, on small

scale, of the same ground should be very much alike, even though made from separate surveys. Mr. Harrison.

It is gratifying that engineers who have visited the Lake Cheesman dam express such high appreciation of the quality of the work. The unusually good conditions for securing this were:

1.—An earnest desire on the part of the management of the Company to have the best that could be built;

2.—The excellent quality of stone available;

3.—The interest taken by the contractors in the work, and an honest effort to build it in a thorough manner;

4.—The intelligent, faithful and constant watchfulness of the engineers and inspectors resident on the work.

S. H. Woodard, M. Am. Soc. C. E. (by letter).—There seems to have been a misconception of the scope of the paper, as shown by the discussions of some of the members and the technical press. The paper was not intended to cover the entire field of arch dams. Such a paper could not properly have been written with the Lake Cheesman Dam as its subject. Mr. Woodard.

The general method of analysis is quite as applicable to the thin arch dam as to such a dam as the Lake Cheesman. In fact it was at first developed for that purpose, and it is believed that it may be made to give much more exact results when applied to thin "pure arch" dams, than when used as in the paper. It solves the old and much discussed question of the effect of preventing perfect arch action at the bottom of a curved dam by fixing it to *terra firma*, in a particularly convincing and satisfying manner, for the results of the analysis may be checked by substituting them in elementary, well-known and proved formulas. One may be assured of knowing the conditions of stress with as great exactness as is possible in any masonry structure; but the details of the analysis are considerably different from that of a case like the Lake Cheesman Dam. Not only that, but the local conditions, such as the shape of the valley, introduce considerable variation in the details. The detailed treatment of a dam in a box cañon would be different from one in the V-shaped valley at Lake Cheesman, although the same method and equations would be used. It did not seem best to take up all the variants of the problems. It will answer some of the questions raised to say that this method of analysis has been tested by applying it to several notable existing arch dams, and the entire evidence from these tests has pointed to its correctness. These computations and tests also indicate that the thin arch dam of moderate radius, properly proportioned in a suitable location, is a thoroughly stable and satisfactory structure.

Mr. Gould proposed a profile for an arched dam, which is based upon the formula, $Q = P\,R'$, in which Q is the unit stress of the arch, P the unit thrust of the water, and R' the radius of the intrados of the

Mr. Woodard.

arch. This formula is strictly applicable only in the case of linear arches, but is approximate for thick arches in case R' is taken as the radius of the extrados of the arch. Had Mr. Gould done this, the profile (Fig. 17) developed by this formula would have been triangular.

He then asks the question: "Can the thrust of the water in the arch dam be considered as being transferred totally to the sides of the valley, etc.?" This question should evidently be answered in the negative. Furthermore, it is not safe thus to consider it, for, by fixing the bottom of the dam, the arch near the base is somewhat relieved of load, but a greater loading is thrown upon the arch of the upper part. This complex distribution of loading usually calls for a considerable increase in the thickness of the upper part of the dam over what would be required if all the water pressure were carried directly to the sides of the valley.

Because the analysis indicates that only a small part of the water pressure is carried by the arch, in the case of the Lake Cheesman Dam, Mr. Gould intimates that there must be discord between theory and practice, and cites the fact that many very light arch dams already built are standing intact. As the same theory applied to the same very light arch dams shows that the arch carries nearly all the water pressure, the same thin arch dams may be cited to show the accord between theory and practice. The point is that the Lake Cheesman Dam is so rigid as a gravity section that there is not much chance for the arch to act.

Mr. Wegmann says:

"If these formulas, however, are to have their full practical value, they must be considered sufficiently reliable to enable us actually to reduce the profile of a 'gravity dam', when curved in plan. Probably few, if any, engineers would be willing to apply Mr. Woodard's formulas to that extent."

Certainly they should not do so, for these very formulas throw the weight of their evidence against such a course. They indicate that, as a matter of economy, unless influenced by local conditions such as at Lake Cheesman, either a pure gravity dam or a pure arch dam will be cheaper, depending upon the width and shape of the valley and height of the dam. Probably many engineers would be willing to apply the method of analysis to the thin arch dam, which is intended to carry all or as much as possible of the load by arch action, especially when it is realized that the results may be checked and the stresses determined as closely as is possible in any masonry structure. However, it is not believed that enough is known about the stress in masonry to make its computation possible to any great degree of exactness, but the thicker the masonry, the greater the uncertainty, and it is probable that the thin arch can be computed more accurately than the thick gravity dam.

Mr. Shirreffs proposes three refinements to be used with the method, of analysis given in the paper. Mr. Woodard.

The first, a refinement upon the use of average moments of inertia and the third, a more refined method of expressing the deflections of the arch, are both points which, as stated in the paper, were considered when the computations were made, but as the approximations there used were on the side of safety and also were quite as well known as the knowledge of the condition of stresses in large masses of masonry, it was considered that, at least for the case in hand, it would not be consistent to make greater refinements in the computations than the assumptions warranted. The writer regrets that he has not had the opportunity, since the publication of Mr. Shirreffs' discussion, to examine his method of computing deflections in detail.

The second modification which Mr. Shirreffs proposes is apparently an error on his part. The first statement of this is not very clear. He says: "The deflections of the arch rings were computed as if due to the constant radius of the extrados of the arches." But, later, in the detailed statement of the arch deflection (page 162), it seems that he proposes to compute the deflection as though the water pressure acted directly upon the center line of the arch. There the thrust in the arch is taken to be $q\ R$, where R is the radius of the center line.

This is an error. The water pressure is upon the extrados of the arch, and its magnitude is a function of the radius of that curve. This water pressure may be transferred to the center line for purposes of computation, but should not be reduced in magnitude by computing it as though it acted directly upon a linear arch coincident with the center line of the dam.

This error accounts for the greater part of the differences shown in Table 6. This table cannot be verified, for the dimensions of the assumed dams for which it was computed are not given, further than that the dams are 180 ft. high. Assuming that the profile shown in Fig. 20 was used, the error in computing Q in Table 6 would vary from 1½ to 20 per cent. If this correction were made, it can be seen that the differences which the table was intended to show must become very small.

Mr. Williams presents a dam which is novel in two respects: First, in the extreme overhang at the top; and, second, in the vertical arch at its base. There seems to be no valid objection to the overhanging top if properly proportioned, and Mr. Williams has advanced many good reasons for it.

The vertical arch feature seems to have been introduced from no reason of economy, but rather for the sole purpose of avoiding the objections that have always been raised against the arch dam whenever one has been proposed or built. These are, first, that the fixing of the dam at its bottom prevents arch action at its base and suggests

Mr. Woodard. the danger that it might be sheared from its foundation. It would seem that the existing examples of thin arch dams would serve to remove this fear. In any case, the Six-Mile Creek Dam is no stronger against shear than it would be if the up-stream side were straight down to rock from, say, Elevation 185. Therefore, it must be assumed that it was not this danger which was intended to be avoided by the vertical arch feature at the base.

The second danger is that the fixing of the base upsets the simple distribution of the load upon the arch, relieving it at the bottom and throwing part of the water pressure from the lower to the upper part of the arch. The only danger in this is that the dam becomes a so-called indeterminate structure, and the loading upon the arch, especially at the top, cannot be found by simple static formulas, but must be determined by some method such as the one given in the paper.

If this matter is considered of sufficient weight, it is theoretically possible to construct the base of a dam with a vertical arch like that of the Six-Mile Creek Dam, which will cause all the horizonal arches above the upper end of the vertical arch to be loaded with simply the water pressure which falls directly upon them. The writer has made no computation of the Six-Mile Creek Dam and will not undertake to say whether or not this end has been accomplished. It would not seem by inspection that to bring the back of the dam straight down below Elevation 185 would increase the rigidity very materially.

However, suppose it be granted that the vertical arch base will accomplish its entire purpose of removing all uncertainty of loading, it becomes a question whether its use does not introduce features much more undesirable than those which it removes.

The only result of bringing the up-stream face straight down to the foundation, besides reducing the intensity of shear, as already mentioned, would be, possibly, to throw somewhat greater loading upon the upper part of the dam. This can be easily met by increasing the thickness near the top. This is usually of little or no additional expense, for there is a corresponding decrease of thickness in the lower part of the dam to offset the increase in the upper part; and, for practical reasons, the masonry cannot be brought to zero thickness where the water pressure becomes zero. The writer's computations show that, for the cases which he has tried, very little increased thickness over the practical minimum is required.

It is a very debatable question if the disadvantages of undercutting the heel of the dam, resulting in the suspended masonry construction and the reduction of the width of joint of the masonry with the rock [this joint consists of only $s\,t$ (Fig. 30), for $t\,a$ is supposed to be an open joint when the dam deflects] does not more than outweigh all the

objections which can be raised against the usual form of construction, especially as it is believed that the more refined methods of computing the stresses in thin arch dams make it possible to proportion them, with regard to their stresses, with a satisfactory degree of certainty. Although the writer has made no computations on the Six-Mile Creek Dam, the profile (Fig. 30) of the 90-ft. dam seems very slender at the top. This may be offset by the use of steel in that portion which was intended to be used, as is intimated on page 192. Mr. Woodard.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 982.

FILTRATION FOR PUBLIC WATER SUPPLIES, WITH ESPECIAL REFERENCE TO THE DOUBLE FILTRATION PLANT AT BREMEN, GERMANY.*†

By Eugen Goetze, Esq.‡

With Discussion by Messrs. W. Kiersted, Howard W. Underwood, William B. Fuller, George A. Soper, T. H. Wiggin, Morris Knowles, Allen Hazen, John H. Gregory, W. R. Copeland, L. J. Le Conte, J. P. A. Maignen, George W. Fuller, William B. Bryan and Rudolph Hering.

The present rules of the German Imperial Board of Health regarding sand filtration comprise a number of important results concerning the building and operation of sand-filter works. They are based upon the experience of many years, collected from the reports of twenty-six filter works. These rules have been printed in full in leading technical publications of the several countries of Europe, and also in America, and reference is made to some of them, in connection with this description of some of the characteristic features of the Bremen filter-works.

The filtration of surface-water is satisfactory when the filtrate is clear, and, as regards color, taste, temperature and chemical composition, is not worse than before filtration, and does not contain more than about 100 bacteria per cubic centimeter, when tested according to a

* Presented at the meeting of November 4th, 1903.
† This paper translated from the German by Rudolph Hering, M. Am. Soc. C. E.
‡ Chief Engineer of the Water-Works in Bremen.

prescribed procedure, including cultivation at 20° Cent. for 48 hours. The bacteriological examination should be made by a permanent officer of the works. The possible pathogenic quality of the bacteria found in the filtrate is not brought into question by this bacterial examination, but the method is merely a means for a delicate discrimination between the results of filtration, and affords a very good control of the operation of the filters. It is based upon the fact that after long experience any filtrate which does not contain the usual low number of bacteria must be viewed with suspicion, or as being of an inferior quality. The figure 100 is merely an approximate limit of the number of bacteria found in satisfactory filtrates of German filters.

The bacteriological examination of every filter should be made daily if possible. If this is not practicable, then daily observations should be made, in accordance with the standard method, at least at the following times: First, for at least two days after starting a filter that has been cleaned, and until the result is satisfactory; second, after the loss of head due to frictional resistance in the sand layer is more than two-thirds of the allowable maximum; third, when the head suddenly decreases; fourth, when high (turbid) water is encountered.

Turbidity observations of the raw water, and, when necessary, of the filtrate, should be taken. They are quite valuable, because, as against the bacteriological observations, they give immediate results, and because the increase of turbidity and the increase of bacteria run parallel. The observation tube, in the apparatus used for this purpose in Bremen, has a length of about 100 cm. Clear water has a transparency greater than this length. Turbid water allows the mark or index to be seen, perhaps, through only 2 to 3 cm. A turbid filtrate, which sometimes results from single filters at high water, shows the index at 40 to 50 cm.

In order to record the degree of clearness to be indicated by this apparatus, samples of water were compared with a series of standards in which clear water is made turbid artificially by adding a known weight of the slimy matter or mud scraped from the surface of dirty filters, after eliminating those portions of it which subside quickly. This gave the results shown in Table 1.

Concerning the preparation of the turbidity standards, it is to be remembered that parts per million correspond to milligrams per liter,

hence the figures in the second column of Table 1 show the number of milligrams of mud added to one liter of water.

TABLE 1.

COMPOSITION OF TURBIDITY STANDARDS CORRESPONDING TO VARIOUS DEGREES OF CLEARNESS OF WATER.

Degree of clearness.	Parts of mud per million of water.
More than 100.	Perfectly clear water.
100	1.7
90	2.2
80	2.8
70	3.7
60	4.8
50	6.5
40	9
30	13
20	22
10	44
5	100

The number of bacteria in the raw water of the Weser, which, normally, is about 1 000 to 2 000 per cubic centimeter, but often is only a few hundreds, increases at high water, such as occurs regularly once or twice in the fall and once or twice in the spring, up to 100 000 per cubic centimeter.

As a preparation of river waters for filtering, preliminary sedimentation in settling basins, operated either intermittently or continuously, may be used with success. The result is equally good in both cases. Smaller settling basins are required for continuous than for intermittent operation. Frost has no deciding influence upon the operation of open settling basins.

It is not generally admissible to reduce the otherwise necessary area of filters on account of the introduction of settling basins. Therefore, the cost, due to the interest on the investment for the settling basins, often exceeds the expected advantage, and it will be more useful to spend the capital saved by the omission of these basins to increase the area of the filters and apply a slower rate of filtration.

In filters which are carefully provided with a sand layer of even grain, and through which the flow of water is regulated according to the best means known to the art, a process of cleaning takes place analogous to that which in Nature produces the highly praised spring water, and justly praised, if, in its formation, it passes through a natural earth filter of sufficient uniformity of material and of sufficient capacity.

If sand filtration is ever justly blamed on account of imperfect operation, the explanation is to be found in errors originating from insufficient knowledge of the nature of filtration. The filtering medium, in the basin filled with sand, is not the sand, *per se*, but the slimy coating (*Schlammdecke*) which is deposited upon and between the grains of the upper layer of sand.

The favorable interaction of bacteria and algæ, the organisms consuming and producing oxygen, should also be mentioned. The bacteria are parts of the slimy coating which are most effective for the filtration. Their work must be done in the uppermost layer, and requires, in the first place, a slow filter velocity. Filtration, for the most part, then, is accomplished in the upper portion of the sand layers, from 20 to 30 mm. in thickness, and the lower layers are thus a safeguard.

In Bremen, and in most cities of Germany, 100 mm. per hour is assumed as the limit of filter velocity; but this rate is reached only during the few days of maximum delivery, when the reserve filters are out of service on account of cleaning. In Hamburg the filter areas have been computed for a maximum velocity of 63 mm. per hour.

If only the uppermost portion of the sand layer is used as a biological filter, decreasing in intensity in a downward direction, and assuming a slow filtering velocity, then there is the guaranty that any bacteria escaping through the upper layers will still be retained in the lower layers.

The foregoing process in a filter for drinking-water, because it is in the main a biological one, has not an unlimited effectiveness respecting the number of bacteria removed. Based on innumerable observations, it is necessary to assume that a ripe filter, provided with a "Schlammdecke," can completely free a raw water of a certain quantity of bacteria and turbidity. In order to get an approximate idea as to what is meant by a certain quantity of bacteria, it will be assumed that there are 10 000 per cubic centimeter in the raw water. The result is a filtrate with a fairly low and constant number of bacteria. If the raw water contains a much greater number of bacteria than mentioned above, the number in the filtrate is no longer constant, but rises and falls in proportion to the number in the raw water. The essentially different behavior of the filter, whether receiving good or moderately good raw water on the one hand, or bad raw water on the other hand, must be carefully distinguished.

This fact is presented most clearly by marking on a diagram, as ordinates, the numbers of bacteria in the raw water and in the filtrate. It is then readily seen that, as long as the raw water has not a very large number of bacteria, the bacteria curve of the filtrate is practically parallel with the zero line, and does not follow the irregularities of the raw-water bacteria curve. After observing this fact for many years, one becomes convinced that, within the above-mentioned limits, the number of bacteria in the filtrate stands in no relation to the number in the raw water. While the bacteria in the raw water have been completely removed, those in the filtrate have entered it subsequently and accidentally.

A filter bed filled with clean sand is not an apparatus which could be called a filter, in the sense of drinking-water purification. Only by the so-called "Schlammdecke" is there formed the filter which is suitable to retain the bacteria and the often still smaller particles of clay. This improved faculty of retention, inherent to a filter that has been in use for some time, is called "ripeness."

From what has been said, the slimy coating originating during filtration must be guarded from destruction with all care. Therefore, interruptions in operation and sudden changes in the filter velocity must be avoided. The increasing accumulation of slime and mud makes it necessary, however, to clean the filter after due time, that is, it is necessary to remove any coating which will not readily pass the water. In doing this, after the water of the filter has been drawn off to a level which is indispensably necessary for cleaning, only the uppermost and visibly the most dirty layer, ranging in thickness from 10 to 20 mm., and sometimes even more, is removed. After the removal of this layer the sand is smoothed down, the filter is supplied from below with filtrate until this well covers the sand, and then raw water is again admitted from above. The filter is not aerated longer than the time required for the process of cleaning.

After such a cleaning the quantitative capacity of the filter is indeed recovered, but the qualitative work, because it depends upon the existence of the slimy coating, is lessened or entirely destroyed. The more care is given to such cleaning the less will be reduction of the effectiveness of the filter relative to quality, because the biological filter, formed by the process of filtration, usually extends to a greater depth of sand, although with decreasing intensity, than that which is

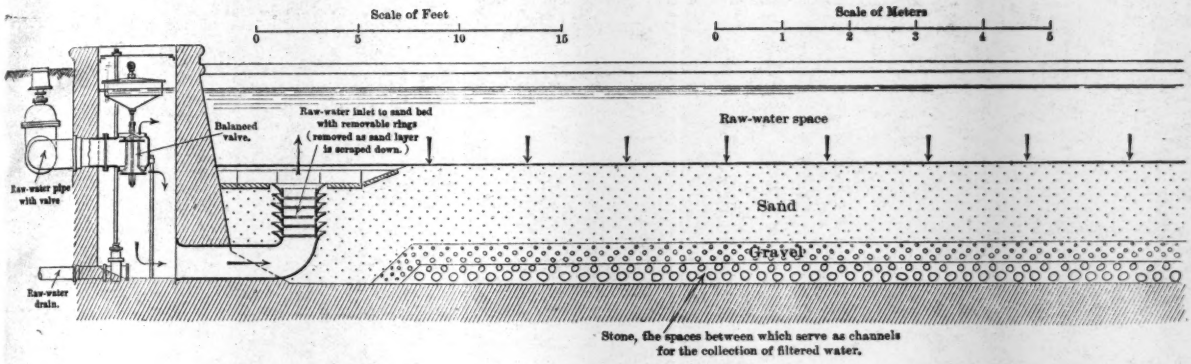


FIG. 1. PIPING CONNECTIONS AND REGULATOR FOR THE APPLICATION OF RAW WATER TO OPEN FILTERS AT BREMEN—INLET.

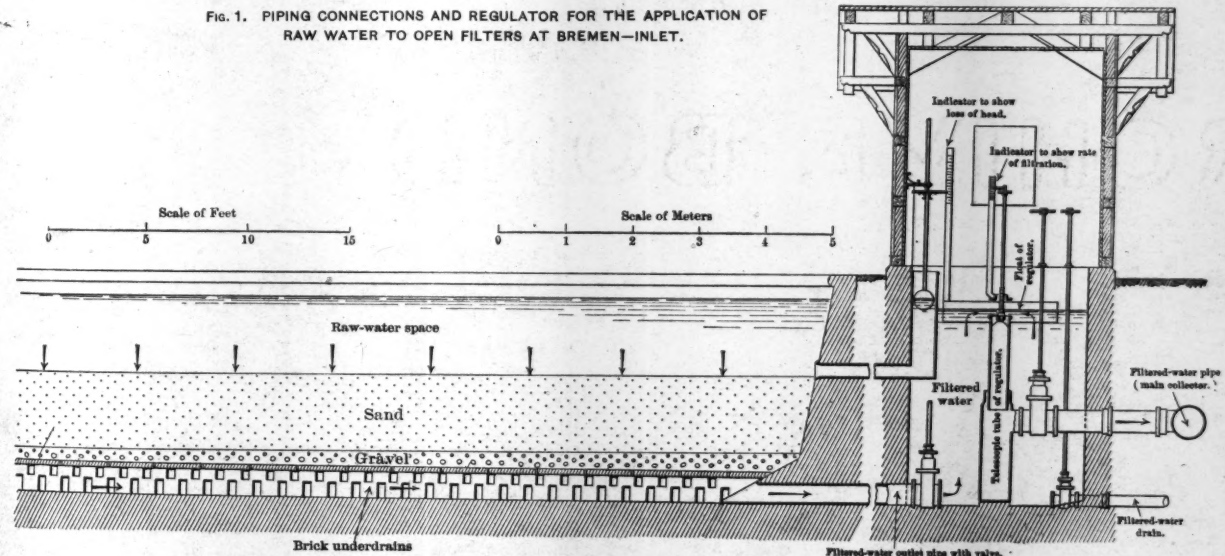


FIG. 2. OUTLET PIPING, GATE-HOUSE, AUTOMATIC TELESCOPIC REGULATOR, AND INDICATORS FOR CONTROL OF FILTERED WATER FROM OPEN FILTERS AT BREMEN.

removed. The coherence of the bacterial mass beneath the removed layer must not be disturbed. It is expedient, therefore, that the men stepping upon the sand be provided with wooden shoes having broad and perfectly flat soles. The removal of sand at cleanings is continued until the remaining depth approaches the allowable minimum, which, for reasons of security, is taken at 400 mm., although the biological filter does not extend as deep. Then the filter is refilled, up to its maximum thickness, with the sand, which, in the meantime, has been washed.

The biological filter, therefore, is impaired by cleaning and entirely obliterated by refilling, and it is necessary to view the first filtrate after these proceedings as being of inferior quality and suspicious. The proof thereof is given by the bacterial examinations, and is, indeed, less distinct after a cleaning than after a refilling, because after the former some ability for proper filtration has remained. The filtrate from a new filter, from one that has been refilled, and from one that has been cleaned or scraped, should remain separated from the water delivered into the supply until the bacterial examinations show favorable results, but at least 48 hours, should a previous observation of the special conditions allow this time thus to be shortened. The latter refers especially to scrapings. The worst water does not appear immediately after starting the filters, but about 24 hours later, according to what may be the first velocity of filtration, because the filter still contains in its lower layers some of the good water of the previous period, and because it was refilled with filtered water from below.

A new or refilled filter, or even a ripe filter, may deliver a poor filtrate during floods.

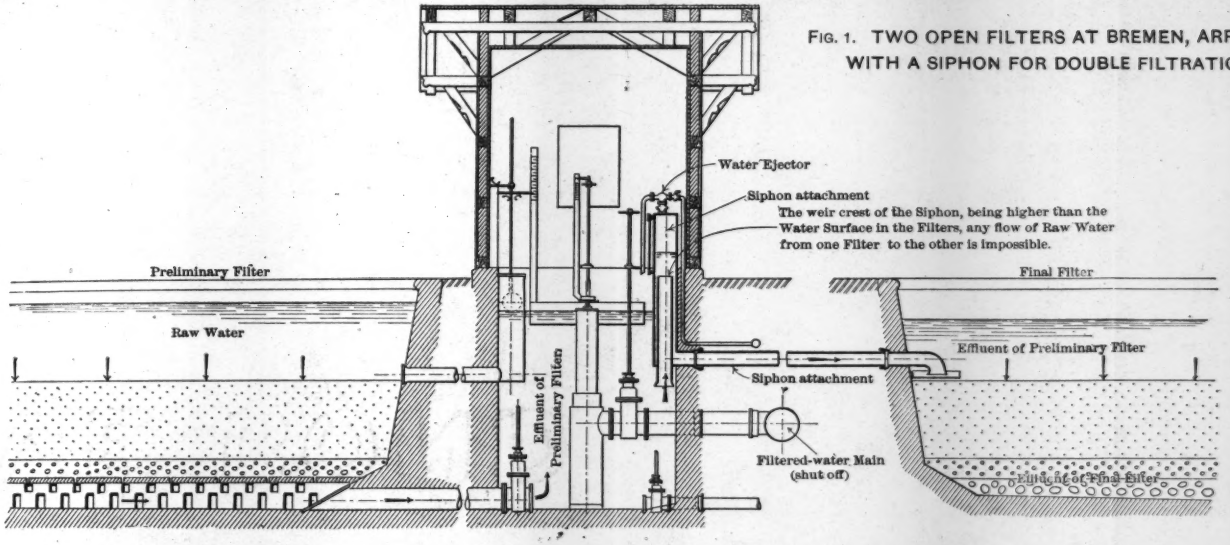
In order to avoid a filtrate that is suspicious or of inferior quality, two ways are possible; local conditions should decide the preference. One way is to allow the filtered water to run to waste. This may be the right way if the raw water flows upon the filters by gravity, and, during all seasons, is in sufficient quantity to permit this waste of a material percentage of the supply. The other way is to filter the suspicious filtrate a second time; that is, to allow it to enter as raw water upon another ripe filter. To do this is never an error, and in many cases it is such an advantage that it almost appears to be a necessity; for instance, when the raw water must be pumped to the filters, or when the available quantity of water is limited and requires saving.

By the second filtration the cost of pumping is saved, but would be lost if the pumped water had to be wasted subsequently. There is a further saving in cost, because the filtrate, which has already passed through a filter once, even though the water does not conform to the highest hygienic requirements, has been deprived of the coarser impurities, so that the second filter receives only a small portion of the impurities which clog the sand layer. It is more economical, therefore, to refilter this water than to purify the same quantity of raw water.

Whether or not double filtration is used, special attention should be paid in every case to the regulating apparatus. Automatic regulation is the best. These devices of the Bremen Water-Works have been gradually improved, and in their present form have been tested for years. Fig. 1, Plate XII, shows the balanced valve and other features appertaining to the regulation of the raw water as it flows to the Bremen filters. Fig. 2, Plate XII, shows the devices for the automatic regulation of the flow of filtered water, which may be described briefly as follows:

In the well into which the filtrate flows is a bent tube, the horizontal arm of which leads to the clear-water reservoir, and in the vertical arm of which, bent upward, is a bronze pipe suspended by floats working telescopically. The filtered water flows into the bronze pipe over an adjustable weir. The depth of water flowing over this weir is constant during oscillations of the water surface, but the overflow edge of the bronze pipe rises and falls with the floats. With this arrangement water flows out of the regulating chamber independently of the absolute height of water in it, but its quantity corresponds to the adjusted height of the weir. It follows that the difference between the surface of the raw water and that in the regulating chamber (in other words, the filter head) will always adjust itself to the necessary quantity. When the filter head is too great, more water flows into the regulating chamber than out of it, and the floats rise. When the head is too small, less water flows into the chamber, and the floats fall until the true head has been obtained automatically. When the surface of the raw water in the filter rises or falls, the water surface in the chamber changes correspondingly. The pressure head, therefore, remains constant, and is independent of oscillations of the raw-water surface. As the clogging of the filter, and therefore the resistance in

FIG. 1. TWO OPEN FILTERS AT BREMEN, ARRANGED WITH A SIPHON FOR DOUBLE FILTRATION.



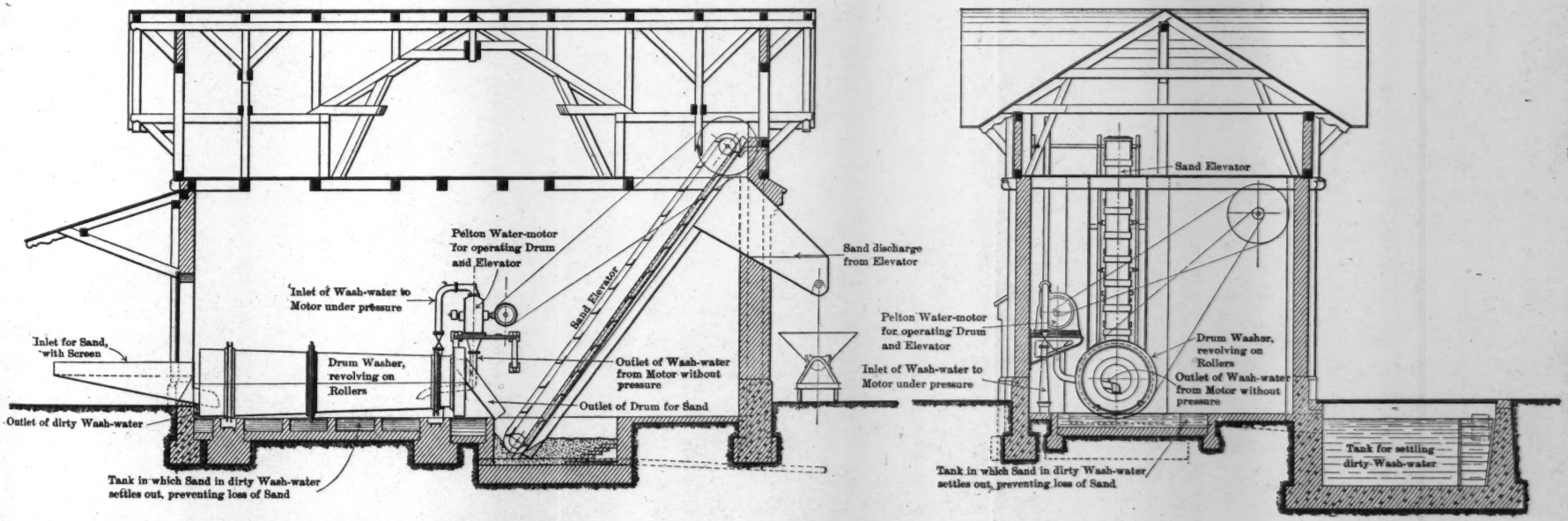
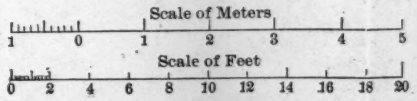


FIG. 2. SAND WASHER AT BREMEN.
DRUM AND ELEVATOR OPERATED BY WASH-WATER.



the same, increases, the pressure head rises uniformly. Both this head and the filter velocity may be read from scales in the regulating chamber. In order to prevent injurious friction, the apparatus in the regulating chambers must be designed and constructed with much care.

The main advantages of these automatic regulators are: That they secure the greatest possible uniformity in the operation of the filters, and, therefore, in their qualitative efficiency; that oscillations in the raw-water level, not to be avoided in the operation of the filters, have no influence whatever upon the regulation of the filters; that the regulating chambers need be visited but rarely; and that labor is saved.

For the second filtering in Bremen, according to the writer's system (see Fig. 1, Plate XIII), the several filters are connected by siphons, one leg of which is in the regulating well for the effluent water of one filter, while the other leg is in the raw-water compartment of the adjoining filter, and the top or head of the siphon is higher than the highest possible water level. By means of a water ejector, a vacuum is created in the siphons. An air-valve makes it possible to unite the inside of the siphons with the outside air, and therefore offers an easy and reliable way of breaking the siphonic action. When the air-valve of the siphon is open, every filter connected with it is a single filter, and is used as such for most of the year. After a refilling or cleansing, the filter is used for a short time as a preliminary filter, that is, the filtrate with natural descent flows into the raw-water chamber of a neighboring filter, the water level of which is kept somewhat lower. The siphons are inserted in such a manner that the filtrate of one filter may be led upon one or the other of the neighboring filters, so as to have a choice in case one of them is out of service or not yet ripe.

It will be noted that at Bremen the filters are open. For conditions where long periods of severe cold prevail, they should be covered.

Illustrative of the effect of double filtration, there is given a series of five tables (Tables 2 to 6), containing results actually accomplished at the Bremen filter-works, and showing the improvement of the filtered water resulting from the second filtration at times when ordinary operations are more or less interfered with by frost in open filters, and when the filters are scraped during periods of high and

turbid water in the river. The more essential details, descriptive of the actual conditions under which the data were obtained, are given for each of the tables.

Following the tables are deductions based on general experience at Bremen.

TABLE 2.

INFLUENCE OF DOUBLE FILTRATION UPON BACTERIAL EFFICIENCY, WITH A HIGH RATE FOR THE FINAL FILTER, WHEN OPEN FILTERS ARE AFFECTED BY FROST.

Date.	Bacteria per Cubic Centimeter.			
	Raw water.	Effluent of Preliminary Filters.		Effluent of Final Filter.
		Filter No. V.	Filter No. IV.	Filter No. III.
1895.				
February 26	650	920	68	23
27	950	1 000	144	8
28	775	280	90	25
March 1	1 500	1 440	70	60
2	925	540	Scraped.	Scraped.
4	1 875	276	73	46
5	2 825	280	114	50
6	3 700	470	240	55
7	2 050	230	176	67
8	3 000	186	196	63
9	2 550	174	224	46
10	2 300	136	134	34
11	1 800	73	84	45
12	2 800	52	86	36
13	6 000	62	82	42
14	11 300	76	110	29
15	12 500	74	150	31
16	8 200	70	126	32
17	.....	76	114	53
18	7 650	46	86	30
19	8 900	46	54	31
20	7 100	50	56	29
21	7 600	49	16	16
22	16 900	48	62	21
23	18 200	40	80	20
24	12 200	28	79	21
25	14 600	46	330	30
26	13 400	61	256	28

Regarding Table 2, it is to be stated that Nos. V and IV are preliminary filters, through which the water was passed first, and their combined area of filtering surface was 2 320 sq. m., the thickness of the sand layer being from 850 to 950 mm. Filter No. III, through which the water was filtered the second time, had an

area of 650 sq. m. and a thickness of sand layer of 1 140 mm. Filter No. V was out of service from the 8th to the 23d of February, on account of excessively cold weather. The rate of filtration in Nos. V and IV, the preliminary filters, was about 60 and 50 mm. vertical drop per hour, respectively, and in No. III, the final filter, about 200 mm. The loss of head in Filter No. V increased during this period from 3 to 50 cm., and in Filter No. IV from 50 to 65 cm. Filter No. IV was scraped on March 2d, after which the loss of head increased from 7 to 52 cm. Filter No. III, the final filter, was scraped on March 3d, and the loss of head during this experiment remained at about 35 cm.

TABLE 3.

INFLUENCE OF DOUBLE FILTRATION FOLLOWING THE SCRAPING OF A FILTER, AS SHOWN BY THE BACTERIAL CONTENTS AND DEGREE OF CLEARNESS OF THE FILTERED WATER.

Date.	RAW WATER.		EFFLUENT OF PRELIMINARY FILTER.		EFFLUENT OF FINAL FILTER.	
	Bacteria per cubic centimeter.	Degree of clearness.	Bacteria per cubic centimeter.	Degree of clearness.	Bacteria per cubic centimeter.	Degree of clearness.
1895.						
March 18.....	7 650	41	36	Clear.	21	Clear.
19.....	8 900	32	Scraped.		17	"
20.....	7 100	40	.....	.....	24	"
22.....	16 900	17	590	Clear.	24	"
23.....	18 200	8	1 720	72	50	"
24.....	12 200	8	1 450	85	71	"
25.....	14 600	11	1 120	91	90	"
26.....	13 400	11	790	81	64	"
27.....	9 800	19	480	87	63	"
28.....	4 600	21	210	95	70	"
29.....	5 300	21	130	95	44	"
30.....	3 400	26	92	Clear.	57	"
31.....	4 500	26	82	"	51	"

Regarding Table 3 it may be said that Filter No. X was scraped on March 19th, and its effluent thereafter was filtered through Filter No. VIII. Filter No. X, the preliminary filter, had an area of 1 160 sq. m., and a sand layer 1 130 mm. thick. The rate of filtration ranged from 65 to 80 mm. Filter No. VIII, the final filter, had an area of 1 310 sq. m., a sand layer 700 mm. thick, and the rate of filtration ranged from 60 to 70 mm.

Each of the two filters used in securing the data recorded in Table 4 had an area of 650 sq. m.

TABLE 4.

INFLUENCE OF DOUBLE FILTRATION AT A TIME OF HIGH AND TURBID WATER, AS SHOWN BY THE BACTERIAL CONTENTS AND DEGREE OF CLEARNESS OF THE FILTERED WATER.

Date.	Raw Water.		Effluent of Preliminary Filter.		Effluent of Final Filter.	
	Bacteria per cubic centimeter.	Degree of clearness.	Bacteria per cubic centimeter.	Degree of clearness.	Bacteria per cubic centimeter.	Degree of clearness.
1896.						
March 2.....	950	53	80	Clear.	25	Clear.
5.....	2 400	56	150	"	76	"
The filters were reversed as to double filtration, that is, the effluent of Filter No. II was turned on Filter No. I (above the sand), instead of into the clear-water reservoir, and the raw water was shut off the latter.						
March 9.....	19 500	3	116	Clear.	36	Clear.
10.....	23 100	3½	295	74	17	"
11.....	28 100	5	780	61	31	"
12.....	16 900	6	616	60	60	"
13.....	19 500	7	610	63	55	"
14.....	10 100	9	620	65	34	"
15.....	11 300	14	185	70	22	"
16.....	9 000	16	155	87	48	"
17.....	4 200	22	95	95	24	"
18.....	4 600	28	76	Clear.	24	"
20.....	1 650	40	38	"	20	"
22.....	1 400	48	49	"	23	"
Double filtration was discontinued, the effluent of Filter No. II flowing to the filtered-water basin instead of to the water space above the sand in Filter No. I; raw water was applied to Filter No. I. These changes were made without any interruption in filtration, in the case of either filter.						
March 23.....	1 000	62	32	Clear.	28	Clear.
24.....	1 000	58	49	"	38	"
25.....	740	57	77	"	49	"
26.....	680	60	80	"	23	"

With the filters used in connection with Table 5, Filters Nos. XI and XII, the preliminary filters, had a combined area of 2 320 sq. m., and Filter No. IX, the final filter, an area of 1 600 sq. m. The rate of filtration in the preliminary filters ranged from 90 to 100 mm., and, in the final filter, from 135 to 150 mm. The loss of head in Filter No. XII increased from 4 to 24 cm., and in Filter No. XI from 3 to 25 cm. In Filter No. IX, the final filter, it was about 30 cm. when secondary filtration was undertaken by it, and this head did not increase during the experiment.

TABLE 5.

INFLUENCE OF DOUBLE FILTRATION AT A TIME OF HIGH AND TURBID WATER, AS SHOWN BY THE BACTERIAL CONTENTS AND DEGREE OF CLEARNESS OF THE FILTERED WATER.

(Two Preliminary Filters and one Final Filter.)

Date.	Raw Water.		Effluent of Preliminary Filter.		Effluent of Preliminary Filter.		Effluent of Final Filter.	
	Bacteria per cubic centimeter.	Degree of clearness.	Bacteria per cubic centimeter.	Degree of clearness.	Bacteria per cubic centimeter.	Degree of clearness.	Bacteria per cubic centimeter.	Degree of clearness.
1896.								
March 2.....	950	53	Scraped.		46	Clear.	12	Clear.
5.....	24 000	56	110	Clear.	160	"	..	"
The filters were reversed as to double filtration.								
9.....	19 500	3	270	58	290	85	56	Clear.
10.....	23 100	3½	660	39	2 840	35	223	97
11.....	28 100	5	860	52	905	50	98	Clear.
12.....	16 900	6	416	53	616	52	84	87
13.....	19 500	7	770	50	670	49	92	95
14.....	10 100	9	290	58	260	63	48	94
15.....	11 300	14	245	72	245	76	52	Clear.
16.....	9 000	16	175	75	220	83	39	"
17.....	4 200	22	90	80	190	94	44	"
18.....	4 600	28	84	Clear.	184	Clear.	25	"
19.....	2 250	32	44	"	106	"	54	"
20.....	1 650	40	48	"	50	"	18	"
22.....	1 400	48	188	"	100	"	28	"
24.....	1 000	58	42	"	61	"	8	"
Second filtration is interrupted								
25.....	740	57	36	Clear.	13	Clear.	29	Clear
26.....	680	60	33	"	51	"	24	"

With regard to Table 6, it is to be stated that, while the water in the river at this period was high, no turbidity was present in the effluent of either the preliminary or final filters. Filter No. VI, the preliminary filter, had an area of 1 160 sq. m. and a thickness of sand layer of 1 120 mm. The sand itself was of fine and even grain, so that the surface of the layer could be made very smooth. Filter No. VII, the final filter, had an area of 1 160 sq. m., and a thickness of sand layer of 485 mm. The sand was of coarse and less even grain, so that the surface was less smooth than in the case of the other filter; consequently, the preliminary filter, with its finer sand, had a thicker "Schlammdecke" than the final one. It will be noted that the preliminary filter worked badly on two occasions, due to cleaning, and also on

two occasions, due to high water, namely, for the several days beginning December 5th and beginning December 17th.

TABLE 6.

Influence of Double Filtration at High Water and after Scraping, as Shown by the Bacterial Contents of the Effluents.

Date.	Bacteria per Cubic Centimeter.		
	Raw water.	Effluent of Preliminary Filter.	Effluent of Final Filter.
		Filter No. VI.	Filter No. VII.
1897.			
November 18	2 800	Scraped.	17
19	2 300	55	9
20	1 750	700	8
21	2 025	280	10
22	1 005	68	5
23	1 325	45	9
24	875	20	6
25	1 080	30	3
26	1 100	22	11
27	950	28	4
29	2 000	75	..
December 1	4 500	94	10
2	9 600	96	9
3	29 000	105	2
4	39 200	180	10
5	38 300	525	15
6	35 600	385	32
7	17 200	165	35
8	7 600	100	30
9	6 400	75	13
10	7 000	75	28
11	7 400	70	21
12	6 400	Scraped.	6
13	10 400	290	24
14	8 000	115	12
15	10 100	115	17
16	9 600	70	20
17	11 100	135	21
18	9 300	240	37
19	6 500	105	29
20	6 600	134	31
22	.....	54	46
27	3 720	49	32
29	2 600	27	21

Referring to the data of Table 6, it is to be pointed out that if the effect of filtration rested only upon a mechanical process, then those bacteria which passed through the thicker "Schlammdecke" and through much finer sand would also have gone through the coarser final filter. That this was not the case is surely an irrefut-

able proof that, not the depth of sand or the size of the grains, but the biological process, caused the final effect.

The fact, noticed in Bremen on every occasion, that double filtration removes the fine clay which had passed the first filter, and is recognizable by an opalescent turbidity, is extremely important. It is well known that in such cases a complete clarification without filtration and without chemical means is impossible. It is also known that in bad cases simple filtration fails. Raw water, which, for instance, was so turbid that the mark on the measuring apparatus could be seen only at a distance of from 2 to 3 cm., and which with single filtration could be seen for about 50 cm., and was opalescent, was improved by double filtration, so that the mark could be seen at a distance of 100 cm., the greatest length of the apparatus, corresponding to perfectly clear water.

Double filtration gives another very instructive example, and shows that under normal conditions the bacteria from the effluent of a ripe filter are altogether independent of the number contained in the raw water, where the filtrate of the preliminary filter is the raw water of the final filter. A perfectly satisfactory filter was used as a final filter for one that had likewise been used for some time. The raw water for the preliminary filter contained on an average about 3 000 bacteria per cubic centimeter, and was changed into a filtrate containing about 40 bacteria per cubic centimeter. The filtrate, therefore, contained about 1.33% of those contained in the raw water. It was then filtered a second time, at a very uniform speed for about 10 days, and contained about 60 to 70 bacteria per cubic centimeter. The second filtrate therefore contained 150 to 175% of those contained in the water delivered by the preliminary filter. Accordingly, the filtrate from the double filter would be worse than that from the single filter, while, in fact, the water filtered twice could only be better. It seems hardly possible to have a clearer proof than this that a constant low number of bacteria, and not a percentage of reduction, should be used as a demonstration of the effective work of a filter.

On the other hand, in considering the operation of a filter with extraordinarily bad water, containing many bacteria and being very turbid, it is found that a constant number of bacteria for the filtrate can no longer be counted upon. In the long, flat curve representing the number of bacteria in the filtrate, there is a very perceptible

irregular movement, a rise and fall analogous to the rising and falling of the bacteria in the raw-water curves. Here the number of bacteria in the filtrate can be expressed temporarily by a percentage of the number contained in the raw water.

No filtrate which is characterized by a noticeable irregularity in the otherwise fairly horizontal bacteria curve should be permitted in the water supply, but should be considered as suspicious water and filtered a second time.

A very noticeable phenomenon accompanying the high number of bacteria contained in the filtrates from flood water, and a very impressive sign of its inferior quality, is the slight turbidity, *i. e.*, the opalescence of the filtrate obtained from very turbid yellow river water. It is not difficult to separate such an imperfect filtrate from the other water intended for distribution, if the cause for such an imperfect filtrate is known, which is generally the case. In general, these periods of opalescent water are noticed after the cleaning of a filter, after refilling it with clean sand, and when the raw water is worse than normal. During these three different periods we should make use of a possibility, hitherto little used, to enable the single filters of a large plant mutually to support each other, in filtering once more the filtrate obtained after single filtration, in case it is not perfect.

It is important, as will be recognized at once, from what has been stated, that the filter which is used as the final filter should have a "Schlammdecke," for otherwise it is not a filter, in our sense. And water having received a preliminary filtration contains only a few of those elements which produce a "Schlammdecke," and is therefore unsuitable to transform a bed filled with clean sand into a biological filter. Therefore, the writer holds it to be necessary, and is justified in doing so by the success obtained in Bremen, that only filters which have been operated with raw water can be used as final filters.

Granted the theory is right, that a ripe filter with a certain filtering velocity can completely purify raw water containing a definite amount of impurities, it might then be said that it should be possible just as completely to purify a worse raw water by correspondingly reducing the filtering velocity. This is true, but a simple computation will show that the possibility of reducing this velocity in practice soon reaches an end.

The area of the filter surface which should suffice for double filtration must be determined from the special conditions of the country and the stream where filtration is required. In Germany, particularly in Bremen, at the Lower Weser, a stream which carries very variable water, and several times both in spring and fall brings very turbid water to the pumping station, the filter area required for the greatest summer consumption with single filtration is sufficient also for double filtration, because during high-water periods in spring and fall the rate of water consumption is much smaller than the yearly maximum.

Generally speaking, for double filtration it is necessary to compute the filter area, first for the maximum delivery with single filtration and then for the possible maximum at times of high water. The larger of the two areas should be chosen. It is correct to introduce into this calculation different filter velocities for single or double filtration; namely, for the single filtration 63 to 100 mm. per hour, and for the double about 100 mm. or more per hour, depending upon special considerations. In Bremen it was found that, with a velocity of 200 to 250 mm. per hour, the final filters occasionally did favorable work.

It will never be necessary to have as much as twice the area for double as for single filtration, and it is always possible to obtain a considerable advantage, even if the filter area is not increased at all, that is, if a second filtration is effected only at those times when the area is really available therefor, as when the consumption is low or after cleaning and refilling; for which purposes, in any event, reserve areas must be available.

By double filtration all is secured that can reasonably be expected from water purification, and chemical assistance is superfluous, but the discoloration caused by humic combinations, which should be distinguished from turbidity, cannot wholly be removed; filtrates from peaty water will retain a slight brownish color unless chemical means are used. The writer has preferred to retain the color if the filtrates were perfectly clear.

In Bremen the water formerly filtered by single filtration contained during every flood 1 000 to 3 000 bacteria, instead of the permissible 100, and was quite turbid besides. Since the introduction of double filtration the city receives, also during floods, a clear filtrate with not more than about 100 bacteria per cubic centimeter. Because double filtration follows single filtration, the final

filters are always in best condition for effective filtration, that is, they are ripe. The entire filter area is very thoroughly utilized, because in summer a large consumption is coincident with a small bacterial content, and the entire filter area can be used for single filtration to secure the full hygienic effect; while during fall and spring the more difficultly filtered raw water is coincident with a smaller consumption, allowing, therefore, on account of the more ample area, a part of the filter area to be used as preliminary and the remainder as final filters. In Bremen, the new area, designed in the year 1892, for single filtration, did not have to be enlarged when in 1895 double filtration was introduced.

All existing filters are in continuous service, including the reserve filters. The difference in the quantity of water consumed is met by a careful change in the filter velocity, as far as the clear-water reservoirs are not capable of compensating for these variations.

The maximum yield of filtered water was 2.4 cu. m. in 24 hours per square meter of filtering surface. The average yield for the year 1900-01 was 1.54 cu. m., and in the year 1901-02, 1.67 cu. m.

The filtering periods between consecutive cleanings range from a minimum, at a time of high water, of about 7 days to a maximum of about 120 days, the average being about 30 to 35 days. During the maximum periods the filters are generally used for quite a time as final filters, during which the loss of head increases very little or not at all.

The loss of head directly after cleaning is about 3 to 5 cm., and it increases until at the end of the period it is about 70 cm.

At each cleaning there is removed from 2 to 3 cm. of sand; that is, in the case of a filter having a surface area of 1 200 sq. m., the total volume of sand removed ranges from 24 to 36, and averaging 30 cu. m. The scraping of a filter of this size is done by ten men in ten working hours.

The sand is not transported from the filter to the sand-washer at the time of scraping, but is moved at some other convenient time. In a similar manner, the washed sand is transported from the washing plant to the clean-sand pile as convenience allows.

Sand-washing facilities, with storage areas, must be provided at filter-works. In Bremen, washers of the revolving-drum type are used. The

wash-water operates a (patented) water motor which drives the drums and the sand elevator. The general arrangement of the plant, which was designed by the writer, is shown in Fig. 2, Plate XIII.

In washing the sand with this newest sand-washer, the services of one man are necessary. He washes in one hour 2 cu. m. of sand, using 7 to 8 volumes of water to 1 volume of sand. This record of the water used includes, not only that used in the washer itself, but also that necessary for turning the wash-drum and in operating the sand-elevator. The pressure of the water used for washing is from 30 to 35 lbs.

The injector system of washing, used at some places in England and America, also in Hamburg, requires 20 to 30 cu. m. of water for 1 cu. m. of sand. The larger consumption of water in this case is due to the fact that the water and sand move in the same direction, while the advantage of the drum washing is due to the fact that these directions are opposite. The advantages formerly accredited to the injector system, namely, that only the wash-water was required for them, that they could easily be decentralized, and that, therefore, there was a saving in the transportation of sand, are now exceeded by the system of drum washing, because the quantity of water which must be brought to the site through pipes is much smaller.

In refilling filters with the clean, washed sand, including the transportation from the clean-sand pile to the filter and spreading it upon the latter, there are required for each cubic meter of sand from 3 to 3½ hours' labor for one man.

The entire operation of the filters, consisting of cleaning, refilling, sand washing, and expense for double filtration, costs in Bremen, for 1 cu. m. of filtered water, annually, 0.166 pf. ($1.52 per million U. S. gallons). The entire annual cost of the water, including maintenance of the distribution system, which for German conditions is quite elaborate, including also interest and sinking fund, which together are reckoned at 6½%, is 8.4 pf. (2.1 cents) for 1 cu. m. delivered into the high-level reservoir, or about $75 per million U. S. gallons. The wages of a common laborer are 3.5 marks ($0.84) per day.

DISCUSSION.

Mr. Kiersted. W. KIERSTED, M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest, particularly as it records the methods of operating sand filters abroad and indicates the character of supervision needed to insure successful results in filtering public water supplies. The description is confined to the slow sand or biological filter, the merits of which has been long and successfully established.

It is generally recognized that the successful operation of any kind of water-purification plant requires skilled supervision, but it is important that the mechanical control of the filter should be so complete and the necessary examination of the water so standardized that the amount of skilled supervision can be safely reduced to a minimum, in the interest of economy of operation.

Long experience with biological filters abroad has doubtless developed structural standards, a permissible range of filter velocity, and approved methods of filter manipulation, which, when duly regarded, insure exposure of the water to the biological processes in a ripe filter long enough to become practically purified and perfectly wholesome, provided the raw water as introduced into the filter is not too turbid. Thus far, efforts seem to have been directed toward the perfection of standards governing the purity and wholesomeness of the output of the filter. Such standards are absolutely essential, of course, and, when once established, they must control, within practical limitations. In order to conform to them in practice, it becomes necessary to vary somewhat the manipulation of the filter, according to the character of the raw water introduced—the more turbid the raw water is, the slower must be the velocity of filtration in order to meet the requirements of the controlling standard of purity. But, as the author has well stated, "the possibility of reducing velocity in practice soon reaches an end," and he proposes double filtration as a means of securing a filtrate of the desired purity without a prejudicial change of filtration velocity, whenever the turbidity and bacterial contents of the water exceed that which admits of conformity to the standard of purity after single filtration.

Doubtless double filtration, as practiced in Bremen, fills the requirements of the situation admirably, as it is necessary for only a brief period of each year. But, if the usual condition of the river water were one of a high degree of turbidity, the writer believes the investment in filters of sufficient capacity for double filtration regularly practiced would prove greater than an investment in a combination of settling basins and single-filtration filters accomplishing equally good results.

Many rivers in the United States furnish water of so great a degree of turbidity for the greater portion of each year that thorough preparation of the water in advance of filtration is required.

Mr. Kiersted.

Thus far, sedimentation in settling basins has proved the best method for preparing the water for filtration. However, with many waters, chemical treatment in the settling basin is imperative in the proper reduction of turbidity, and by its assistance it is believed the turbidity may be so reduced that single filtration will afford a perfectly wholesome water after the filter is in proper condition for biological work.

The writer believes that a standard should be adopted for the turbidity of a water admitted to a biological filter, as in this way alone can a uniformity of filtering velocity and of manipulation be secured which enables a filter to deliver uniformly an acceptable product. He would make filtration altogether a refining process, and would meet the fluctuation of turbidity of the raw river water in the settling basins and there modify the manipulation and treatment of the raw water to suit its changeable physical characteristics.

Incidentally, there will be a reduction of bacteria during the partial clarification process in the settling basin, but the writer would fix no standard for bacteria reduction during this process, as this is obviously the work which the biological filter is expected to perform.

The adoption of such a standard for the raw water would result, among other things, in minimizing the amount of skilled attention required in the operation of the water purification plant, because the conditions surrounding the operation of the filters would be more nearly uniform, day after day, and the variation of the physical characteristics of the water would be readily observable in an open settling basin and controlled accordingly. Moreover, it is believed to be cheaper to remove the accumulation of sediment from a settling basin than from a filter.

It is admitted, however, that some skilled attention is required to operate a settling basin successfully, particularly where circumstances require the use of a coagulant. But, in this regard, methods can be so thoroughly standardized that the skilled supervision can be made a comparatively easy matter.

Mr. Underwood.

HOWARD W. UNDERWOOD, JUN. AM. SOC. C. E.—In his description of the Bremen filters, Mr. Goetze has called attention to many interesting points connected with their operation. One, which is especially interesting, as it is a method differing from that used in America, is the transportation and washing of sand, and it will, perhaps, be of interest to describe the methods used at the Philadelphia filters, and trace the progress made there.

Two systems, known as the Upper and Lower Roxborough filters, have been completed and in operation for some time at Philadelphia. At the Lower Roxborough filters, for some time after they were started, the dirty sand was removed from the beds in wheel-barrows

Mr. Underwood.

and washed at a convenient time. In March, 1903, pressure and discharge pipes were placed in the filters, and since then the dirty sand has been transported from the bed directly to the washers by a portable ejector under water pressure. After passing through the washers, the sand is transported by water to the clean-sand pile.

The ejector is simply a hopper in which a horizontal jet of water passes through a sprayer pipe and flushes the sand down toward the nozzle. Ordinary heavy fire-hose is used to connect the ejector with the permanent iron pipes, which extend through the center of the filter. The dirty sand is not only removed from the bed by the ejector, but is also washed and delivered to the clean-sand pile, without extra labor. The force required, therefore, is only large enough to keep the filters scraped, as the men work at this all the time.

No additional water is required, as the ejector does the greater part of the washing, and not all the hoppers in the washer are required, as compared with washing directly from the pile.

At Lower Roxborough, with one ejector and a pressure of from 55 to 60 lbs., from 4 to 7 cu. yds. of sand per hour are removed and washed by four men, using from 2 800 to 3 000 galls. of water per cubic yard; or, in other words, from 14 to 15 volumes of water to 1 volume of sand.

At Upper Roxborough, with two ejectors and a pressure of from 80 to 90 lbs., from 14 to 18 cu. yds. of sand per hour are removed and washed by seven men, using from 2 200 to 2 400 galls. of water per cubic yard, or from 11 to 12 volumes of water to 1 volume of sand.

The total cost of scraping the filters, removing and washing the sand, including all changes in the pipe, and the cost of water, varies from 40 to 60 cents per cubic yard.

The total filtering area at Lower and Upper Roxborough is 8.25 acres, and eight men are constantly engaged in scraping, removing and washing the dirty sand, which amounts to about 1 000 cu. yds. per month.

This demonstrates the effectiveness of the ejectors for cleaning a filter quickly, and also shows a great reduction in cost, as compared with removal by wheel-barrows and washing later.

As to the relative cleanness of the sand: Before the ejector was installed, five hoppers were used, and the turbidity of the average scrapings, about 40 000 to 50 000 parts per million, was reduced to about 2 000 parts per million. At Upper Roxborough, at present, the ejector and one hopper are used, and the turbidity of from 40 000 to 50 000 parts per million is reduced to about 500 parts per million. As the sand is delivered from the ejector pipe, it has a turbidity of from 1 200 to 1 500 parts per million, and with proper means of removing the dirty water, it is believed that the ejector will give sufficient washing.

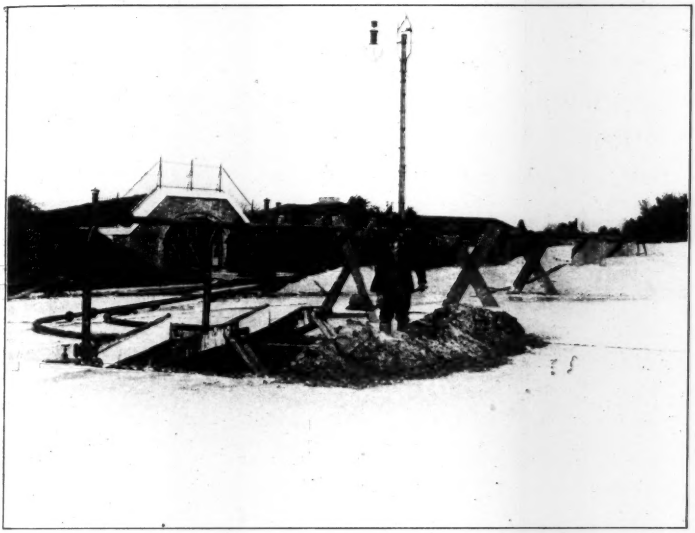


FIG. 1.—GENERAL VIEW OF SAND-WASHER AND PIPING AT UPPER ROXBOROUGH FILTERS, PHILADELPHIA, PA.

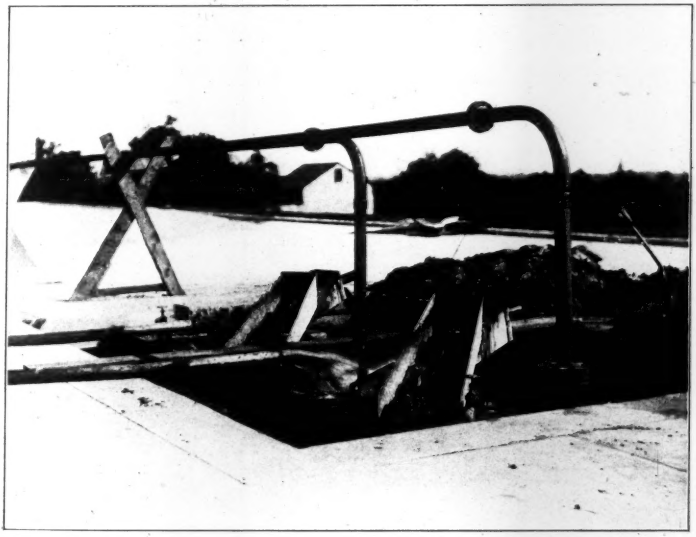


FIG. 2.—PIPING, TROUGHS, ETC., OF SAND-WASHER, UPPER ROXBOROUGH FILTERS, PHILADELPHIA, PA.

The average length of run on the Roxborough filters is about 35 days, and a rate of about 3 000 000 galls. per acre per day is maintained. From 1¼ to 1½ ins. of sand are removed at each scraping. Mr. Underwood.

WILLIAM B. FULLER, M. AM. SOC. C. E.—The speaker has been very much interested in the description of the working of the Bremen filters, and especially in the ingenious method used to secure the benefits of double filtration of the water coming from filters when first put in commission. Mr. W. B. Fuller.

The principle of refiltering the first part of the run of a newly scraped filter, where it would be uneconomical to allow this filtrate to run to waste, is certainly in line with the best practice of filter operation, but the speaker would hesitate to recommend the accomplishing of this object in the manner described by the author.

One of the first tenets in the design of slow sand filters in America has been to make them as near fool-proof as possible, and to take especial care that there are no channels through which, either by accident or design, raw water can be run to the filtered-water conduits without passing through a sand layer.

It is feared that the existence of a by-pass may prove too great a temptation to a foreman desirous of making a record for long runs between scrapings, and the resulting economy of maintenance, and it is customary on this account to provide only one by-pass between raw and filtered water for use in case of accident, this by-pass being strongly guarded with locks and seals so that it cannot be opened without the knowledge of those highest in authority.

In the design shown, the seal of the siphon may be accidentally or designedly maintained after changing to single filtration, and in this manner raw water could be delivered directly into the filtered-water conduit.

The additional first cost of construction is also a bar to the adoption of the method described, as it may, and often does happen, that periods of greatest turbidity are coincident with periods of greatest consumption.

In America no definite rule has been laid down for guidance in the operation of filters, but it is customary to assume a sufficient purification if the average of the effluent from all filters gives a bacterial content of less than 100 per cubic centimeter.

If the principle is recognized, however, that the filtrate from each filter shall have less than the above bacterial content before entering the filtered-water conduit there will be no additional first cost in the double filtration proposed, as in any case provision would have to be made in the design for additional filters to take the place of those in the process of ripening.

In a design with which the speaker is now connected the principle

Mr. W. B. Fuller.

has been recognized that it is the best practice not to turn the first filtrate from a newly scraped filter into the filtered-water conduit, and provision is made for the wasting of this water to an effluent-drain reservoir from which reservoir all water required for transporting and washing sand is to be pumped. This water can also be pumped back into the raw-water conduit if the sand washing and transportation does not require its use, and this cost of pumping, which is very small, owing to the small lift, is believed to be a good investment as an insurance against the improper use of by-passes.

Should the water from single filtration show turbid in the effluent, the remedy recommended by the speaker would be a preliminary filtration through a mechanical filter run at a high rate and without coagulant.

The author appears to be a great champion of automatic regulation, having automatic regulators on both the raw-water inlet and on the filtered-water outlet. This may be, and probably is, the best method for small plants receiving raw water under pressure, but the speaker is opposed to their use where they can be avoided.

The taking of the raw water from a large reservoir of constant level, and the control of the effluents from a large number of filters from a central house by hand regulation appears to him to present a more nearly fool-proof combination than automatic regulation.

The author is wrongly informed as to the volume of water required for washing sand by the injector system, as the poorest ejectors in use at any of the plants known to the speaker do not require more than 15 volumes of water to 1 of sand, and the newest types compare favorably with the volumes required by the washer described by the author, with the additional advantages of transporting the washed sand to any distance.

Mr. Soper.

GEORGE A. SOPER, ASSOC. M. AM. SOC. C. E.—This paper describes in a clear and instructive manner how its author has solved one of the most common and unsatisfactory problems of water purification. It is easy to filter the water of the Weser, at Bremen, under ordinary circumstances, but, at times, it may become very difficult. In fact, it is impossible to remove the impurities by a single filtration, and it becomes necessary to pass the water a second time through the filter bed.

In the purification of river waters, it may be said that such conditions are the rule in many of the Central and Eastern States. The frequency with which the periods of difficulty are met, and their duration and severity, are the important influences in deciding the means for assisting the filters. If exceptionally roily water is to be encountered only three or four times a year, as at Bremen, such an arrangement as that described by the author may offer the best solution. If the water is more often turbid, and the impurities to be removed are excessive

in amount, it may be desirable to use large settling basins, as are common in America, or the suspended particles may be precipitated with chemicals, as in the United States and in some places in Holland. If the streams are comparatively short and the intervals of turbidity brief, the plant may be shut down entirely until the worst of the impure water has disappeared, as is practiced in England. Finally, if the effluent of a filter is objectionable only because of its bacterial contents, the water may be sterilized, for example, with ozone, as has been done successfully in Germany. Mr. Soper.

In determining in a precise way the factors upon which a choice of the most suitable means of preparing a water for filtration depends, and in treating it after filtration, lies an important field for further investigation.

To many it will be a new idea that a water may need any other treatment than filtration, for there is a general impression, outside of the engineering profession, and among some persons in it, that if a water has been once filtered it must of necessity be pure. In frankly acknowledging the limitations of the filter at Bremen, and in affording means for understanding wherein these limitations lie, the author has performed a useful service.

Those who have had occasion to design or operate a filtration plant, with a water which is usually clear but sometimes turbid, know the fallacy of supposing a filter to be capable of heavy work. To those who have a practical knowledge of water purification, it is evident that the capacity of filters of all kinds to handle suspended matters is limited, and that the conditions under which they can be operated safely must be known thoroughly and kept in mind constantly if results are required which are to be satisfactory analytically.

In relation to this it may be interesting to recall the definition of a filter given by the pioneer of water purification in America, the late James P. Kirkwood, M. Am. Soc. C. E. He called them "screens of the greatest delicacy." To-day, Mr. Kirkwood's definition is illustrative rather than accurately descriptive. Filters are more than screens, yet they are like delicate screens in being able to remove fine particles of suspended matter and at the same time unable to handle a great deal of it.

Nothing in these remarks should be taken to indicate that the speaker is opposed to the use of filters in the purification of water supplies. Engineers are forced to recognize the efficacy of filters in many ways and are not disposed to underrate their value. They have accomplished inestimable good in protecting communities of all sizes, from small villages to cities of the first class, against cholera, typhoid fever, and other infectious diseases. But it is not correct to assume that the whole art of water purification rests in the filters. In the speaker's opinion, a broader view of this subject should be taken.

Mr. Soper. It should be admitted that, for successful working, water filters of all kinds must be handled with great care; and that either very fine or very coarse material, or a great deal of suspended matter of any kind, soon puts them out of service.

It was once thought that the structural elements of rapid water filters could be designed so that a simple combination and adjustment of them would fit them to any water. This view has had to be modified. Filters are not as elastic as they were formerly thought to be. The leading question to-day is not how to adapt filters to different waters, but how to adapt the different waters so that they can be handled by the filters.

The author has well said that the product of purification, and not the percentage of removal, should be the standard for judging the success of a plant. In this country an efficiency of 99% in removing bacteria is generally accepted as satisfactory evidence of a filter's capacity, in this direction, and if the numbers of bacteria found in filter effluents do not exceed 200 per cubic centimeter, as determined by American methods, it is often contended that they should be considered satisfactory. This may or may not be all that should be demanded. Much depends upon the character of the impurities to be dealt with. If the filter is built to purify a water which is unpleasant to the sight and taste, and there is no special need of removing bacteria, one may be satisfied with a high percentage of removal and comparatively large numbers of bacteria in the effluent. If, on the other hand, as most often occurs, the purification plant is intended to remove color, turbidity and sewage bacteria, the standard by which the operation of the plant should be judged must be the amount of impurities in the water after treatment. As long as rivers and streams are regarded as natural sewers, into which all manner of wastes may be thrown without restriction, plants for the purification of drinking water derived from these streams should be built so as to remove practically all the bacteria. In accomplishing this end, it will be necessary to take into account many processes which were formerly considered to be preliminary or supplemental, and not indispensable to purification by filtration. The so-called preliminary processes include sedimentation and precipitation by chemicals. The processes by which water may be purified after passing through a filter are not as well known. The author has indicated one of them.

There seems to be good reason to believe that it will not long be beyond the range of practical expediency to sterilize public water supplies which it is not practicable to purify adequately by a single filtration. Many instances of late have served to call attention to sterilization. The question has been discussed before engineering societies in America, France and Germany, and before the International

Congress of Hygiene and Demography. There are a great many official reports, monographs and journal articles at hand, in which the views of many scientific investigators have been expressed in favor of sterilization. Mr. Soper.

The sterilization of public water supplies is not a new idea. It was suggested by a member of the American Society of Civil Engineers in 1894, and the same gentleman wrote a paper on the subject before this Society at a later date. The process advocated was distillation. Basing an estimate, on the cost of distilling a supply of several million gallons of water per day, on an evaporative efficiency of 14.3 lbs. of water per pound of coal, it has been stated that the annual cost of supplying distilled water to a community of 40 000 people would not be in excess of $1 *per capita*. Since this estimate was given, improvements have been made in methods of evaporation, which would probably reduce the cost of producing very large volumes of distilled water. A plant erected for the United States Government at Fort Jefferson, Dry Tortugas, with a daily capacity of 90 000 galls., has developed on official trial an efficiency of 22.8 lbs. of water per pound of coal, the water in this case being salt.

The present outlook is not unfavorable for the use of chemical germicides for the sterilization of water on a large scale. The germicides are added in minute quantity, accomplish their result quickly and are removed from the water without in any way interfering with its attractiveness or hygienic qualities.

At first sight, the idea of killing the bacteria in water before drinking it appears to carry with it serious objections, but, upon considering the matter carefully, it will be evident that these are founded upon mere popular prejudices similar to those held against the use of chemical coagulants in rapid filtration. Arguments brought against the wholesomeness of sterilized water and food have, in the average case, but little scientific foundation. Sterilized food is a common article of commerce, and the enormous sale of canned goods, all of which are sterile and a very large proportion of which have been treated with chemicals to help flavor, color or preserve them, is increasing. Within recent years, a large demand has grown up for sterilized water and ice, so that there is hardly a city in this country, of 10 000 inhabitants or more, which has not a plant for the preparation of these conveniences.

T. H. Wiggin, Assoc. M. Am. Soc. C. E.—This paper presents to the speaker four more or less separate ideas which have an important bearing on the present design of slow sand filters. Mr. Wiggin.

1.—The main idea of the paper—that double slow filtration will produce an effluent having a constantly low number of bacteria at all seasons, and not merely a relatively low number of bacteria, as does single filtration.

Mr. Wiggin.

2.—A suitable *Schlammdecke* is only obtainable (in a reasonable time) by the application of raw water.

3.—At least 48 hours filtrate should be refiltered or wasted, being likely to contain an excessive number of bacteria.

4.—Ejector washers are not as economical as revolving washers.

The ideas will be referred to in different order, beginning with the third.

Of course, there is nothing new in the fact that the water filtered first is high in bacteria. The Germans have simply arrived at a higher standard than the Americans. Filtration, here, is in its infancy, and public sentiment is hardly educated to demand any filtration. At the same time it would add not a very large proportion to the cost to pump up and refilter, or waste, as may be most economical in the particular case, the first 48 hours' flow from newly scraped filters. If the shortest period between scrapings were ten days, it would take, at most, a plant of about one-quarter more capacity. Time will create a demand for this in this country. Perhaps engineers, even now, could carry it through in cases where filtration was finally adopted. The effect, on the average quality of the filtrate, of using the first part of runs, evidently depends on the proportion of newly washed filters, and, ordinarily, the visible effect in a large plant would not be great.

Regarding the fourth idea, the speaker believes the Philadelphia results, and, still further, the results of some other plants now under construction or being planned, will change Mr. Goetze's views. He is evidently unaware of the present development whereby sand is washed and transported at the same time direct to its place in another filter, without storage or rehandling. The author does not give details of cost of washing or moving sand from the beds, and it may be that some of our present plants having ejector washers would compare favorably in the cost of the equivalent items for ejector washing. The total cost per million gallons ($1.52) is not so much less than the cost at Albany, for example, as to account for the great difference in wages. Only a detailed account of cost showing what items are included in Mr. Goetze's cost would enable this question to be decided.

Referring now to the first or main idea, of double slow filtration, there can be no dispute that this accomplishes "all that can reasonably be expected from water purification," at least, until the world has moved along a good deal. The question is, can it be accomplished more cheaply by another way? The speaker thinks there are very few cases in this country, like Bremen, in which the same area that is commonly allowed for single filtration will do for double filtration at times of high water. The only consumption tables the speaker has at hand are those of the Metropolitan Water District, in Massachusetts. The amounts for 1902 are as follows:

Millions of gallons per day.	Millions of gallons per day.	Millions of gallons per day.
Jan., 118	May, 106	Sept., 108
Feby., 117	June, 110	Oct., 106
March, 108	July, 108	Nov., 105
April, 103	Aug., 107	Dec., 125

It is evident that, in Boston, there is no great difference between the consumption in periods of high and low water. The author gives 63 to 100 mm. per hour, as the velocity in the preliminary filter, and 100 mm. or more, as the velocity in the final filter, or, say, a ratio of $\frac{3}{4}$ to 1. If the consumption in periods of high water were $\frac{1}{1+\frac{3}{4}}$, or 0.57 of that for which the filter plant were designed, then it would be possible to use double filtration at periods of high water without increasing the size of a plant designed for single filtration at the minimum rate mentioned. In cases where the consumption is practically constant, evidently about three-fourths more filter area would be required to use double filtration on the basis of the above rates.

The tendency of American ideas seems to be toward rough preliminary filtration at rates and by devices suggested by the American or rapid system of filtration, though generally without the use of chemicals, with subsequent slow sand filtration at unusually high rates. This method would result in a comparatively cheap plant. The method of the author, doubtless, would not compare with it in cheapness. The question is, whether the American method can be relied upon to approach the German in effectiveness. The results of the Philadelphia and Harrisburg experiments would throw great light on the question, and their publication will be awaited with much interest. Even if this new method does not give quite as good results as double slow filtration, American cities, as a rule, are not ready for the additional cost which the author's method would entail.

Regarding the second idea, that raw water is required for a *Schlammdecke*, this has a direct bearing on any double system of filtration. If the preliminary filtration is pretty complete in removing turbidity, it would seem reasonable that it might be desirable to have provision for admitting raw water to the newly scraped filters. A comparatively small pipe would do this, because only a small proportion, say one-tenth, of the filters would be using raw water at once. Information on the necessity of this provision, for the new American system of double filtration, from Philadelphia and Harrisburg, will be looked for with great interest.

Evidently, a little excess area in the final filters would be required for this method of applying raw water to furnish a *Schlammdecke*, as a slow rate for the raw water in single filtration would be necessary; and a still further excess would be needed if the first filtrate were

Mr. Wiggin. thrown away or admitted to the other filters, by the author's method, by pumping, or by any other method.

Mr. Knowles. MORRIS KNOWLES, M. AM. SOC. C. E.—This paper is interesting, for several reasons, and affords an opportunity of comparing European and American practice. It is particularly fortunate that there has been added to American literature the views of a European engineer on the subject of filtration, and information about a filter plant located in a country where so much attention has been and is being devoted to this subject.

It is interesting to note that the "number of bacteria in the effluent" is the accepted criterion of good work, rather than the "percentage of removal." This is to be commended, and should be accepted universally by students of filtration. A step further, and one which is receiving more recognition in America, where the size of the plants warrants expenditures for such studies, is the investigation into the pathogenic quality of the bacteria in the effluent, at least, in so far as this is indicated by the presence of *B. Colon Communis*, the representative bacterium of sewage contamination.

It is not always true that high numbers of bacteria, above 100, in the effluent from a filter, indicate a water unfit to use for domestic purposes, if the large numbers in the raw water are due to decaying vegetable matter, and occur with a sudden rise in the stream. On the other hand, the character of the effluent, at times, may be damaged seriously and the health of the people endangered, even though the number of bacteria remaining in the effluent is not larger than occurs without causing comment; although, frequently, however, the presence of *B. Coli* is accompanied by large numbers in the effluent, but, perhaps, not more than are just as often present without such dangerous association. In the case of high rates with sand filters, the *B. Coli* test is particularly useful, and is more delicate than a general count in determining the suitability of the effluent.

Too great emphasis cannot be placed upon keeping in touch with the workings of the filter day by day. Of course, this is much more important with a rapid or mechanical filter, depending upon the use of a coagulant for its uniformity and stability of results; but it is also true, although to a less degree, with the slow sand filter, even though a lack of attention to the details of operation is not usually attended with such disastrous results as in the former case. For a small filter plant, where a technical laboratory man cannot be employed constantly, and where chemical and bacterial samples must be sent to outside laboratories for analysis, and thus such tests are made infrequently, the observation of the turbidity of the filtered water affords a ready and prompt means of determining its suitability for drinking purposes. Not only if mud, or finely divided clay, is passing through the filter, is it likely that more bacteria than usual will be present in

the effluent, but it is also true that, after people are accustomed to a clear sparkling water, there is a decided dissatisfaction if turbid water be furnished, and it is proper and right to cater to this desire for a bright water, of pleasant appearance, even if a turbid one does not always indicate a less healthful water with more bacteria in it. Mr. Knowles.

There is another peculiar factor, accompanying a large quantity of mud in the applied water, which the speaker has noticed several times, notably in Philadelphia, while in charge of the filter testing station, and for which he has not yet been able to present a satisfactory explanation; that is, that after a continued spell of muddy applied water, when the effluent begins to show signs of turbidity, the number of bacteria in it increases above the normal amount. This was true, even though the numbers in the raw water (probably due to dilution) were not always increased excessively, although generally somewhat larger than usual. In other words, high numbers in the raw water, when clear, were not accompanied by correspondingly high numbers in the effluent, but, on the other hand, high numbers (perhaps not as great as in the former case) if occurring with muddy water will almost always be accompanied by a great number in the effluent.

In regard to the use of settling basins, the speaker cannot agree with the author that their use will not render it admissible to cut down the number of filters; nor does he believe that, in case of muddy water, settling basins can be dispensed with and more filters used, so as to operate at a lower rate. First, assuming that the applied water is muddy only at infrequent intervals, the use of the lower rate, in a case of having additional filters, or the use of the settling basins, will only be required a part of the time; thus, it is best to have the least expensive construction, *viz.*, settling basins, which will cause lower fixed charges on that part of the plant in use only a part of the time. In the case of water being very muddy all the time, some means of coagulation may become necessary, which is accomplished best in settling basins. If settling basins of less capacity (probably about one-half) are required for continuous use, as compared with intermittent, quiescent operation, it can hardly be said that the result is the same in each case for an equal expense.

It is to be noted that the rate of filtration in German cities is much lower than in America, the maximum being lower than the average in most cases here. Recent investigations in America have shown that rates as high as 5 000 000 and 6 000 000 galls. per day to the acre, formerly thought quite excessive, can be used in many places with good results, when the water is not too muddy. It must not be thought, however, that it will be safe to run such high rates with all waters, and first a study must be made to ascertain the results from high rates for all seasons, before deciding upon its application in any given case. But the fact remains, that the German rate is about one-half or two-

Mr. Knowles. thirds as much as has been recognized as allowable for most waters in this country, and this is especially noticeable if the bacteria and turbidity given in the tables are a reasonable indication of the general average conditions abroad.

The author has carefully called attention to the fact that, in his practice, very high numbers of bacteria in the applied water are accompanied by poor results in the effluent. In the tables presented in the paper, this appears to be true of the effluent from the first filtration, although the highest numbers in the raw water are not as high as occur frequently in the waters of American rivers. It has been noticed in this country, however, that the character of the effluent remains practically the same, and low in the number of bacteria, even though those in the raw or applied water run up to hundreds of thousands, provided the river is not too muddy, as mentioned previously. In order to make this clear and show that the effluent of sand filters, when they are in good condition, is not proportional to the numbers in the applied water, the speaker had prepared two diagrams showing the results obtained at Pittsburg, Pa., and at Lawrence, Mass., with experimental filters. Both diagrams show numerous instances when excessive numbers in the applied water are not accompanied by proportionately high numbers in the effluent, and in several cases are fully as low as the average of those which occur ordinarily in the filtered water. On the Pittsburg diagram, the turbidity of the river water is also plotted, and, in many instances, the high numbers in the effluent are preceded by muddy applied water. The injurious effects of the winter weather upon these exposed open filters stands out very clearly, and several instances of disturbances, due to cracks in the walls, and manipulations occasioned by the experimental nature of the filters, are to be noted. On the whole, however, the general height of the effluent line is below 100 on the scale, except in severe weather, and it does not always follow the line representing the applied water. Scrapings and disturbances of all kinds are more noticeable on the Pittsburg diagram than on the Lawrence one, as the latter shows distinctly that the effluent remains in good condition practically all the time. It should also be noted that the Merrimac River water does not have such high bacterial counts and does not contain as much mud as the Allegheny.

In reference to the depth of sand which in Bremen is allowed to remain in place before refilling, *viz.*, about 16 ins., which is generally less than is allowed in this country, the writer does not state whether, at the time of such refilling, a large quantity of sand, say from 6 to 10 ins., is removed, or else shoveled over and placed on top of the fresh, or newly washed sand. This is the practice in this country, and is done so that there shall be no subsurface clogging, as will occur if a portion of dirty sand is left below a clean body of washed sand.

Mr. Knowles.

TABLE 7.—RESULTS WITH EXPERIMENTAL SAND FILTERS AT PITTSBURG, PA.

Showing Number of Bacteria per Cubic Centimeter in Effluents for each of the First Five Days after Scraping; Compared with the Average for the Run (Omitting the Record on the First Day).

FILTER NO. 1.								FILTER NO. 2.							
No. of run.	Number of days run.	Number of bacteria in effluent. 1st day.	2d day.	3d day.	4th day.	5th day.	Average of run.	No. of run.	Number of days run.	Number of bacteria in effluent. 1st day.	2d day.	3d day.	4th day.	5th day.	Average of run.
2	54	64	44	43	67	....	22	2	17	58	92	50	16	25	40
3	37	175	84	35	26	26	33	3	42	88	....	81	28	36	19
10	28	48	115	70	....	39	65	4	44	130	130	....	21	19	55
11	34	58	43	31	31	25	45	14	28	77	84	88	61	105	66
12	36	110	84	73	78	125	135	15	26	68	75	63	63	93	110
13	32	385	560	100	75	75	80	16	16	105	120	190	90	68	75
14	32	290	280	67	55	58	55	17	18	93	100	73	58	58	66
16	31	50	72	58	42	47	45	19	19	96	145	60	49	37	66
17	34	52	330	110	54	28	53	20	10	225	240	125	110	77	105
19	12	250	215	110	105	66	85	32	37	90	375	96	330	....	92
25	33	96	240	....	230	120	93	33	39	70	90	....	65	49	120
27	16	....	330	140	135	14	105	36	25	87	....	64	60	16	31
32	39	....	345	120	19	42	30	38	37	67	235	....	38	31	24
35	10	....	66	92	38	17	40	40	11	60	360	....	....	89	110
36	10	63	....	94	67	....	59	41	12	60	....	36	84	27	43
38	20	540	....	240	125	140	97	42	12	....	115	46	....	62	64
45	12	....	60	89	....	37	110	51	21	79	105	240	350	....	100
47	21	120	370	320	160	155	92	52	14	47	16	43	22	....	115
48	18	15	31	25	24	22	33	53	16	73	92	35	22	....	29
49	19	10	....	48	20	22	28	54	21	61	201	....	120	120	51
50	17	215	145	105	....	92	82	56	17	150	....	57	69	40	68
51	14	....	48	16	36	17	30	57	15	140	57	17	18	9	37
52	16	....	81	35	12	32	24	58	16	165	44	20	42	....	65
53	12	115	97	78	22	60	45	59	19	67	42	....	22	29	28
54	13	....	39	25	47	23	30	60	18	165	58	12	15	....	23
55	18	14	....	80	17	23	50	61	17	....	62	17	19	12	24
56	17	750	125	64	13	....	30	62	28	29	280	130	31	....	56
57	18	43	....	6	21	22	16								
58	25	69	80	195	94	....	63								
Averages	....	160	160	88	62	52	58	Averages	....	94	135	74	72	50	62

In regard to wasting the water from a filter immediately after cleaning, the speaker believes that in most cases this is an unnecessary refinement, unless the conditions are known to be unusual and likely to produce bad results. There is presented, in Table 7, the results of the first few days of several runs of the Pittsburg experimental filters, which show conclusively that the number of bacteria during the first 24 and 48 hours after scraping are not dangerously higher

Mr. Knowles. than those for five days, or indeed for the average of the whole period. In the cases given in the table, the rate of filtration for the first 24 hours after scraping ranged from 500 000 to 3 000 000 galls. per day, so that the applied water would appear in the effluent from 8 to 10 hours after starting.

In many of the instances where the numbers during the first two days are high, the applied water was more than usually muddy; in fact, the filter was put out of service, due to the quantity of mud, and quickly started again, which dosed a raw filter surface heavily and which need not occur in the operation of a municipal plant with the usual reserve area.

The same story is told by the results obtained with the Lawrence, Mass., city filter, as tabulated by Charles G. Hyde, Assoc. M. Am. Soc. C. E., and the speaker.* It may be seen here that the difference in the character of the effluent before and after scraping, even with most of the filter surface disturbed, practically does not amount to anything. It is important to dwell upon this point because of the frequent and erroneous statements made, that the effluent from a sand filter must be wasted for some little time after scraping. All such precautions, while duly conservative, diminish the yield and filter capacity unnecessarily. It is much better, in the speaker's opinion, if such precaution is necessary, to provide a reserve filter area, which is needed for many other reasons, and depend upon a short period of rest after filling and then start with slow accelerations in the rate.

The foregoing statement can be applied to double filtration, especially in reference to waters containing no higher numbers of bacteria than appear in the tables presented by the author. For, while the effluents of the preliminary filters cannot be considered satisfactory, as given, it appears unexplainable to the speaker why better results, or as good as found in the final filter, should not be obtained with a raw water no worse than is shown. The author states that a double filter is never an error. While this may be true, from the scientific and bacterial standpoint, the monetary phase must be considered, and many places, now having water improved by filtration, would never have thought it possible to incur the expense of purification if double filtration, meaning almost double the cost, were necessary. In this country, with the lavish use of water, occasional demands for maximum quantities, frequently 25 or 30 % greater than the average, are just as likely to come at a time when a sudden downpour, or rainy weather, has produced a muddy stream. While it is generally true that turbid waters occur in the spring and fall, the most difficult waters to treat are frequently seen in the winter, or in the midst of summer weather, when the demand for water to prevent freezing of pipes (in the first season) and for sprinkling purposes (in the second)

* *Transactions*, Am. Soc. C. E., Vol. XLVI, p. 304.

has brought the use to the maximum. Thus double filtration in America means twice the filter area. Mr. Knowles.

The statement, that double filtration has always succeeded in providing perfectly clear water, is a gratifying one; but the speaker wonders whether sedimentation, either with or without coagulation, as the necessity demands, with one filtration, would not have produced the same result at a less total cost. The use of a coagulant is an expense solely at the time of application, while the fixed charges for additional filters are a constant source of drain. It is surprising, to say the least, referring to Page 233, third Paragraph, that any one should think of double filtration for a water which has come from the first filter with only 40 bacteria per cubic centimeter.

The use of automatic devices, both on the inlet and outlet, is to be commended. This is especially true as the novelty of such works for the purification of water is disappearing, and mankind is proverbially accustomed to grow careless with familiarity and less likely to be watchful of changes. The size, also, of some of the recent plants being built in this country, renders it imperative that the number of gates to be attended to, and the number of things to be done each day, shall be lessened as much as possible.

It is interesting to note that the allowable loss of head in Bremen is about 27 ins., while, in this country, 48 ins. is deemed advantageous. While 16 ins. is the minimum depth of sand allowed, 24 to 30 ins. is the usual figure in this country. The filters here commonly have 48 ins. of sand, when filled to maximum depth, and also the same depth of water over the surface.

The sand removed at each scraping is somewhat in excess of that in this country, being from $\frac{3}{4}$ in. to $1\frac{1}{8}$ ins., the lower figure being the same as at Lawrence, Mass., the highest published figure in this country for a municipal plant, the average at the Pittsburg experimental filter plant being about 1 in. The speed of scraping, also, is rather less than in this country, and, from the statements, other operations seem to be in harmony with this. From this, it may be reasoned that the low cost of all operations, *viz.*, $1.52 per million gallons, is due, more than anything else, to the low cost of labor, which is only about one-third to one-half of that in this country.

The various costs and quantities are given in Table 8, and compared with the data relating to some American filter plants, most of which have been taken from *Transactions*.*

The statements relative to sand washing are quite instructive, especially to American engineers, for the speaker does not know of the use of the drum washer, at least at filter plants in this country. Statistics as to the cost of washing per cubic yard would be of interest, although it may be expected that, with the quantity given as being

* *Transactions*, Am. Soc. C. E., Vol. XLVI, p. 258.

Mr. Knowles. washed per hour and the number of men in attendance, this cost will be low. It seems as if the services of more than one man would be necessary to bring sand to the washer, to attend to the operation of the machine and to carry sand away to the pile of clean material; all of which items are in this country charged to washing. The author is in error, however, in supposing that 20 to 30 volumes of water are needed with the ejector type of washer, for in more than one place in this country 10 volumes are all that are used, and, in some instances, suffice for all the operations of washing, from carrying the sand from the dirty pile to the washer and finally delivering to the pile of clean sand ready for use.

TABLE 8.—SUMMARY OF DATA FOR SEVERAL SLOW SAND FILTERS.

Items.	Bremen, Germany.	Lawrence, Mass., City Filter.	Lawrence Experimental Filters of Massachusetts State Board of Health.		Albany, N. Y.	Mount Vernon.	Pittsburg Experimental Filters.
			3-*B*.	8-*A*.			
Time included.....	1900–1902	1893–1900	1893–1899	1893–1899	1899–1900	1896–1900	1897–1899
Average rate, in millions of gallons daily.	1.70	1.25	2.35	2.77	2.60	1.70	2.90
Average period between scrapings, in days.....	32	29	26	29	25	24	23
Average yield, in millions of gallons daily, between scrapings..	55	36	62	80	66	41	68
Scrapings per year...	11	13.3	13.8	12.6	15	15	17
Average number of inches removed at each scraping.....	0.98	0.75	0.49	0.53	0.71	0.61	1.05
Cubic yards per acre removed at each scraping.....	132	100	65	71	95	82	141
Hours of labor for each scraping and wheeling out	336	228	.....	.....	176	.....	.....
Sand washed, in cubic yards per hour.....	2.61	0.58	.....	.....	0.64	.....	.....
Volume of water to sand	7 to 8	10	.....	.....	12.5	.....	.....
Hours of labor sanding per cubic yard	2.5	1.5	.....	.....	1.3	.....	.....

The drum type of washer does not lend itself readily to the method of transmission of sand by water to and from the washer, which is the line in which the most advanced progress is being made in this country to-day; even now, sand can be taken from the piles in the filter without the use of wheel-barrows. The nearest approach to this, with the drum washer, within the speaker's knowledge, is the use of the Greenway washer, at the East London Water-Works (which, by the way, washes 5 cu. yds. per hour), where the Engineer, Wm. B. Bryan, M. Inst. C. E., has installed two ejectors, to be used with the washer, one at the

filter to receive the sand as scraped and send it to the washer, and one at the outlet of the washer to convey the clean sand to another filter. This arrangement, however, means additional expense, simply for throwing capacity, which might just as well be used in washing and is thus utilized with the ejector type of machine. Mr. Knowles.

ALLEN HAZEN, M. AM. SOC. C. E.—The Society is to be congratulated upon receiving a paper from Mr. Goetze. In the last few years a number of American water-purification plants have been described before it, and the speaker thinks it most profitable to consider the developments, in another country, toward the same end, namely, the complete purification of water, but which, as the paper shows, have taken somewhat different lines from those followed in recent American plants. Mr. Hazen.

The speaker has had the pleasure of visiting the Bremen filters and of meeting Mr. Goetze, and seeing his most ingenious and satisfactory methods of water purification, and, therefore, can appreciate the paper more fully.

The Weser rises in the Hartz Mountains and flows northward through the flatter country before reaching Bremen. The character of the water is much like that of the Hudson at Albany.

The size of the water-shed of the Weser and the urban population upon it, in comparison with some American streams, are given in Table 9.

TABLE 9.

City.	River.	Drainage area above intake, in square miles.	Urban population per square mile in 1900 (towns with more than 4 000 inhabitants).
Philadelphia	Schuylkill	1 915	98
Bremen	Weser	15 100	About 70
Lawrence	Merrimac	4 634	69
Albany	Hudson	8 240	46
Philadelphia	Delaware	8 186	30
Pittsburg	Allegheny	11 400	16
Washington	Potomac	11 476	7

The data as to the drainage area and urban population upon the Weser are less accurate than for the American streams. It would be appreciated if the author could furnish more exact data upon these points.

The Weser is like the Hudson, in that it carries a little peaty matter. Its turbidity seems to be about the same. It is certainly more turbid than the Merrimac at Lawrence, probably less turbid than the Allegheny at Pittsburg, and very much less turbid than the Schuylkill and the Potomac. The drainage area is a little larger

Mr. Hazen

than that of any of the American streams mentioned. The urban population per square mile gives it a place above the average, but not at the top of the list. The closeness with which the raw water approaches the waters of these American streams gives an added interest to the methods of purification used.

It is particularly interesting to note that the Bremen filters are open. Mr. Goetze takes this as a matter of course, and does not mention cold weather as one of the causes of reduced efficiency at Bremen, although he speaks of the necessity of covering in more severe climates, about which there can be no question. It is interesting to note that the winter temperature at Bremen is substantially the same as that of Philadelphia and Pittsburg. Evidently, Mr. Goetze, who, of course, is familiar with the operation of covered filters, for there are many such in Germany, does not regard this winter temperature as one requiring covers for filters.

The speaker believes experience shows that, for this and warmer climates, covers are to be regarded as a luxury rather than as a necessity; that is to say, with good management, open filters can be operated with as good results as covered filters, and without any great increase in the cost of operation. In fact, in some respects, labor-saving machinery for sand handling is more readily used with open than with covered filters.

Probably the most important advantages of covers, in climates of the severity of those mentioned, is the possibility of employing the men every working day in the year, with the economies which come from their constant and regular employment.

The most striking feature of the paper is the method of partial double filtration. The author notes that when filters are scraped, that is to say, when the top layer of dirty sand is removed, as it must be occasionally, to maintain the service, the efficiency of the filter afterward is much decreased, and after the sand has been reduced in thickness, and washed sand is replaced to restore it to its original depth, the effluent thereafter for some time is of worse quality than that obtained otherwise. This decrease in efficiency has been noted everywhere where sand filters have been studied closely.

To see how much influence these factors have had on the results at Albany, all the data bearing upon these points, which include the daily bacterial counts before and after scraping and refilling since the plant was put in operation, have been compiled and averaged. The figures were taken from the very complete records kept by Mr. Leonard M. Wachter, Chemist and Biologist of the plant, and for them the speaker is indebted to him.

The averages of these figures are given in Table 10.

From the summary given in Table 10 it appears that the decrease in efficiency following scraping during the eight warmer months of the

year is very slight. Often, no decrease whatever is observed. This has been particularly true in the latter part of the period. On the general average for the whole period, the increase in the number of bacteria after scraping, although noticeable, is not large enough to be serious. Mr. Hazen.

TABLE 10.—AVERAGE NUMBER OF BACTERIA IN EFFLUENT FROM ALBANY FILTERS BEFORE AND AFTER SCRAPING, AND BEFORE AND AFTER REPLACING SAND, FROM SEPTEMBER, 1899, TO SEPTEMBER, 1903, INCLUSIVE.

	256 scrapings during the 8 warmer months (April to November).	115 scrapings during the 4 colder months (December to March).	Fourteen refillings.
Third day before	44	194	68
Second day before	48	213	52
Last day before	62	272	68
First day after	91	386	498
Second day after	74	741	570
Third day after	82	1 100	444
Fourth day after	91	1 468	461
Fifth day after	82	1 312	586
Raw water	14 500	74 800	25 200
Mixed effluent	60	596	131
Average efficiency of whole plant for the same periods	99.58%	99.20%	99.48%

In the four colder months, the conditions are quite different. After scraping, the number of bacteria reaches, on an average, a maximum of about seven times the number present before scraping, and this high number persists for a number of days, and the additional number of bacteria is so considerable that, apparently, the number of bacteria in the mixed output for the four months is two or three times as great as it would be if the efficiency could be kept up to that obtained just before scraping. As it is, the filters, during the winter months, with very high numbers in the raw water, have given an efficiency of about 99.2 per cent, If the decreased efficiencies following scrapings could have been eliminated entirely, the average winter efficiency would have been 99.6 or 99.7%, or equal to the actual average summer efficiency.

The decrease in efficiency following the refillings with sand is quite as marked, but, owing to the much smaller number of such periods, the effect upon the average output is comparatively small. Some refillings are accomplished with no more disturbance than summer scrapings; at other times, much higher numbers are found in the effluents following the operation. The results are so erratic as to suggest strongly that many of the bacteria found in the effluents after refillings are due to growths in the underdrains and sand during the period

Mr. Hazen

of a week or more that the filter is out of service. As far as this is the case, the high numbers are rather misleading, for bacteria which occur in this way are entirely harmless.

The method proposed by Mr. Goetze is to filter a second time the effluents containing more than the normal number of bacteria. This is an entirely logical proceeding, and probably the best way of accomplishing the result. Double filtration has been used in a number of cases. The works at Schiedam, in Holland; those of the Grand Junction Water Company, at London, and, more recently, those at Zurich, are among the best known. Two sets of filters have been used in these cases, the first-filters and the after-filters being at different levels and of somewhat different construction.

At Altona, double filtration was introduced in a somewhat different form. The rules of the German Government required effluents to reach a certain standard of purity, and required that when effluents were not of this quality they should be wasted. The filters at Altona were on a hill, from which the city was supplied by gravity. The preliminary pumping was thus more expensive than usual; and to waste the effluent represented a greater increase in the operating expenses. Centrifugal pumps were installed to pump that part of the effluent not of satisfactory quality back into the filters, to be filtered over again. This arrangement was an improvement; but Mr. Goetze's siphons, with the filters all at one level, made the use of centrifugal pumps unnecessary, and, in this respect, was a distinct advance over the Altona arrangement. His process is patented in Germany, but the speaker believes it is not patented in the United States.

When the speaker examined Mr. Goetze's apparatus at Bremen, and the records of operation obtained with it, in 1896, he was impressed with the idea that a most substantial step forward in the art of water purification had been made; and if he could have been sure of securing the same faithfulness in operating the plants with which he has been connected as can be generally depended upon in Germany, he would, undoubtedly, have made use of this device in several cases. His reason for not using it was this: The siphons work equally well forward or backward, and, with careless or vicious attendants, they may be operated thus; that is to say, raw water can be run into the filtered-water pipes through them, and, in fact, this will happen if the siphons are started with the water levels in their normal condition. It is only by lowering the water on one of the filters, and making the conditions to that extent abnormal, that they can be operated as intended.

It has been the speaker's custom to design filter plants practically or entirely without by-passes. At Albany, for instance, there is no possible pipe connection for sending raw water to the pure-water pipes. The only way that this could be done would be to dig up

the sand layer somewhere, an operation of some difficulty, which would require some time, and which would be apparent to anyone who happened to be present and who understood the construction of filters. Mr. Hazen.

With the siphons placed as under the author's patent, it would be possible at any time for an attendant to send raw water into the filtered-water pipes, and this could be done during the hours of the night and absolutely no trace of it would be left in the morning. The attendant might do this because he was too lazy to make the changes in the gates necessary to maintain the supply of filtered water.

In the past the speaker has been unwilling to take the chances of an unauthorized use of a by-pass, but systems of filtration are now coming into use which, in much higher degree, depend upon the faithfulness of the attendants, and the use of the siphons is certainly as safe a procedure as the application of a coagulant, where the efficiency of filtration depends upon it.

If the use of siphons in the Goetze system would increase the winter efficiency of a filter plant from 99.2 to 99.6%, as the results obtained at Bremen, in connection with a study of the Albany figures, indicate, it may be considered seriously whether the advantage is not enough to counterbalance the possibility of the objectionable use of siphons; but it should be remembered that if the filter were by-passed, even for one night, the number of bacteria sent to the city would be much larger than the number eliminated by the second filtration during the whole course of a winter.

Practically, the speaker is inclined to think that the procedure of starting filters at a lower rate than the nominal, if carried out with the same increase in filtering area that would be necessary for double filtration, would prove a very effective treatment, and would perhaps increase the quality of the output to a corresponding extent. The first effluent has never been wasted at Albany, and, as far as the speaker's observation and inquiry go, wasting the first effluent has been a theoretical, rather than a practical, matter. It has been advocated by certain persons, but, practically, it has never been followed systematically in any water-works, as far as the speaker knows, because it is both better and cheaper to start the filters at reduced rates, and to make the rates as low as the conditions permit, and as are shown to be advantageous.

The discussion serves to emphasize the fact that filtration produces a relatively, but not an absolutely, pure water. The standard of purity of public water supplies has increased in the past, and there is every reason to believe that it will increase further in the future; and that the methods which appear satisfactory to us to-day will seem as inadequate a generation hence as some of the older methods seem to us to-day.

Mr. Hazen. The art of water purification is bound to advance to meet these requirements, and methods of purification more efficient than those in use to-day will be required, and will be developed gradually.

Mr. Gregory. JOHN H. GREGORY, ASSOC. M. AM. SOC. C. E.—The speaker has been very much interested in this paper, and also in the discussion which it has brought forth, particularly that relating to the washing and transporting of sand. Mr. Underwood's discussion on the transporting of sand at the filters in Philadelphia and the results obtained has been of especial interest, inasmuch as the method in use there is the one which was recommended by the speaker.

The author, evidently, does not favor the ejector type of sand-washer, but, in this country, this type is in almost general use, and, in the speaker's opinion, is here to stay. It may be of interest to give briefly a little of the history of the ejector washers in this country, and also of the development of the method of transporting sand by water.

The first washer of which the speaker has any record was built at the filter at Lawrence, Mass., early in 1895, and was first put in use on June 17th. This washer consists of four hoppers fitted with horizontal nozzles. In 1897 and early in 1898 Morris Knowles, M. Am. Soc. C. E., who was then a member of the Lawrence Water Board, began experimenting with different arrangements for transporting sand, and, as a result of these experiments, a portable hopper or ejector was built and put in regular use in July, 1898. This ejector, however, was not, and has not been, used for transporting sand from the filter, but for transporting sand to the washer after it had been wheeled up to the bank beside the filter.

At the filters at Poughkeepsie, N. Y., in 1895, and also again in 1896, a single-jet washer was used,* according to Mr. Charles E. Fowler, the Superintendent. In November, 1897, a double-jet washer was used. No drawings of either of these washers are given, but, from the description, presumably, they are of the ejector type fitted with horizontal nozzles. From the jet in the bottom of the second hopper of the double washer, the water and sand were transported about 130 ft. to a tank over one corner of the filter. The dirty water flowed off from the top of the tank and the sand was discharged on the filter through a valve in the bottom. In August and September, 1897, it became necessary to provide some new sand for the old filter at Poughkeepsie. This sand was delivered in scows alongside the dock near the filters. Mr. Fowler designed a movable ejector which could be lowered into the scows and into which the sand could be fed. The sand was lifted 30 ft. by this ejector and transported 630 ft., through a 4-in. cast-iron pipe, to a tank, the dirty water overflowing

* In a paper on the Poughkeepsie filters, before the New England Water Works Association, in 1898.

from the top of the tank and the sand being discharged through a valve in the bottom, and then spread over the surface of the filter. Mr. Gregory.

In 1902 it became necessary to purchase additional filter sand, and, for transporting the sand from the dock to the filters, a method similar to that used in 1897 was adopted. In this case, however, the sand was first hoisted 20 ft. by the buckets ordinarily used for hoisting coal, and then fed into an ejector. From this ejector the sand was transported 600 ft., through a 6-in. cast-iron pipe, to the filters.

In 1897, when Allen Hazen, M. Am. Soc. C. E., designed the Albany filters, he included two washers of the ejector type. These washers each have five hoppers fitted with vertical nozzles. The details of these washers were worked out, by William B. Fuller. M. Am. Soc. C. E., and the speaker, from rough sketches made by Mr. Hazen.

It was while the speaker was connected with the Albany filters, and also after reading the Poughkeepsie report for 1897, that he first became interested in the washing and transporting of sand. The subject of transporting was very fully discussed by Mr. Fuller and the speaker at various times from 1897 to 1899, and numerous methods were worked out in enough detail to know that they were perfectly feasible. From their studies they concluded that the ejector method was the most promising, the only difference of opinion between them being in the design of certain details.

In the fall of 1899 the Albany filters were started in operation before the construction work was entirely finished, and for a period of several months the speaker had the opportunity of studying in detail the transporting and washing of sand as carried on there. As is well known, the sand is wheeled out from the filters to the court and then put through the washer. After watching these operations he became convinced that by adopting the ejector method a great improvement could be made in the transporting of the sand, together with a material reduction in the cost of operation.

Before leaving the Albany work, the speaker made the suggestion that while it might not be possible at that late date to make the additions and changes necessary for installing the piping, so that ejectors could be used in the filters, it would be possible in the court to transport the sand to the washer and take it away to a storage pile by means of water.

Early in 1900, George I. Bailey, M. Am. Soc. C. E., who was Superintendent of the Albany Water-Works at that time, designed and had built a small portable ejector which was used for transporting the sand to the first hopper of the washer. The operation of this ejector was very satisfactory, not only in decreasing the cost of washing, but also in increasing the capacity of the washers. The

Mr. Gregory. washed sand, however, was not taken away by water, but was wheeled away, as had been the common practice.

Shortly after the speaker became connected with the filtration of the water supply at Philadelphia, in the spring of 1900, he was asked by George S. Webster, M. Am. Soc. C. E., who was the Chief Engineer at that time, what he considered the best means of handling the sand to and from the filters, and he replied that he would use the ejectors or handling it from the filters and that he felt that it might be possible to handle it in a similar manner back into the filters. At that time, he felt that it might be better to provide stationary ejectors in the filters ready for use at once, but on further study he came to the conclusion that portable ejectors would be more practicable.

The problem of designing a system for transporting sand from the Philadelphia filters was such a large one that the speaker felt that there ought to be some experimental data on the subject, and in August of that year he suggested that an experimental machine be built for obtaining such data. After some delay, a machine was built and a series of experiments made, from July to November, 1901, under his direction. This machine consisted of two hoppers fitted with horizontal nozzles and a long discharge pipe. It is to be regretted that the results of these experiments have never been made public. While they were not complete, by any means, they threw considerable light on the subject, and demonstrated amply that sand could be transported about 400 ft. with a lift of about 13 ft., and with an economical use of water.

The sand-washers at Lower Roxborough, which Mr. Underwood has described, were designed before, and those at Upper Roxborough after, the experiments just mentioned were made.

At the time the speaker suggested making the experiments, he felt that it would be possible to go a step further than just transporting the sand to the washer; that is, that the sand could be taken from the machine, the last hopper acting as an ejector, and transported by means of water back into another filter which was being refilled with washed sand.

It would not be practicable, however, simply to dump the sand on the filter, allowing it to spread itself, and letting the water run away to a drain, as there would probably be considerable trouble in the subsequent operation of the filter.

When sand is discharged by means of water, and allowed to spread out, the finer particles separate from the larger ones and the sand is found to be stratified. Furthermore, the water will be more or less turbid, and, as some ponding would take place, a certain amount of sediment would be deposited. Should sand be replaced in a filter in this manner, subsurface clogging would be sure to take place when the filter was put in operation. To obviate this difficulty the speaker

believes that he is the first to suggest that the sand should be discharged into a tank, somewhat similar to the one used by Mr. Fowler at Poughkeepsie, but which could be moved from point to point in the filter. The water would be allowed to overflow at one end and would be carried away by a pipe to the drain provided for the purpose. The sand would be discharged through a valve in the bottom of the tank directly into wheel-barrows and then wheeled away and spread. Mr. Gregory.

It was found, in the experiments at Philadelphia, that wet sand could be loaded successfully into a perforated wheel-barrow through an opening in the bottom of a tank filled with sand and water, provided the effective size of the sand was not very small. At present, in Philadelphia, sand is not being transported back into the filters by means of water, but the speaker understands that the method is soon to be tried.

At the experimental testing station in Philadelphia, in the summer of 1901, a sand-washer was built for use in connection with the experiments which were being made on the filtration of water. This washer is an exact duplicate of those at Albany, having been made from the same patterns.

At the filters in Mt. Vernon, N. Y., in 1897, a washer was designed and built by Mr. A. H. Kneen, the engineer in charge there. This washer consisted of four hoppers fitted with horizontal nozzles. In 1900 a second washer was built similar in general design to the first one.

At Cincinnati, Ohio, while the experiments were being conducted on the filtration of water, a washer was built, early in 1898, under the direction of George W. Fuller, Assoc. M. Am. Soc. C. E. This washer had four hoppers, and was fitted with horizontal nozzles.

At Milford, Mass., a small filter plant was designed and built in 1902 by Leonard Metcalf, Assoc. M. Am. Soc. C. E. At Mr. Metcalf's request, a sand-washer and a portable ejector, for use at that plant, were designed by the speaker. This washer consisted of only two hoppers and was fitted with horizontal nozzles. The speaker regrets that, at the present time, he is unable to give any figures relating to the transporting and washing of sand at this plant.

The speaker believes that, when an ejector is used in connection with a washer, two hoppers in the washer are ample for washing the sand. In fact, he thinks that satisfactory results may be obtained with one hopper. When sand is transported a considerable distance through a pipe, it gets a good washing, and if, when discharged into the hopper, the dirty water can be carried away quickly, the sand will be found to be nearly clean. The additional washing which the sand receives in passing through the hopper should be sufficient to make it clean enough for all practical purposes.

In the designs for the proposed filters at Pittsburg, Pa., made in the

Mr. Gregory. winter of 1901 and 1902 by Mr. Hazen and Mr. Knowles, provision was made for transporting sand from the filters to the washers by means of ejectors. These washers were to have consisted of two hoppers fitted with horizontal nozzles.

At New Haven, Conn., filters are now being built by A. B. Hill, M. Am. Soc. C. E., for the New Haven Water Company. The general designs for this work were furnished by Mr. George W. Fuller, and it was at the suggestion of Mr. William B. Fuller and the speaker that he recommended the transporting of sand from and to the filters by means of water; and to them was entrusted the design of the apparatus for transporting and washing the sand.

In the design of this plant, in which the filters are covered, a radical departure from previous practice has been made. There will be no so-called court, but instead, a long covered pipe gallery with the filters ranged on either side. In this gallery will be placed the apparatus for washing sand. No sand will be stored, as it is proposed to transport it directly from the washer back into some filter which will be always kept in reserve for receiving washed sand. Only one washer will be provided, consisting of two hoppers fitted with horizontal nozzles, and mounted on trucks so that it will be movable. In operation, this washer will be placed half way between the filter from which the sand is being transported by means of ejectors and the filter which is being refilled.

In this filter will be placed a movable receiving tank which can be supported from the roof of the filter at various points. The dirty water will overflow into a chamber in one end of the tank and will be carried away through a light, spiral, riveted-steel pipe. It is expected that at this plant a considerable reduction in cost of maintenance will be obtained.

It should be stated that the method of operation just outlined is mainly applicable to covered filters, but it is equally applicable to open filters in a climate where the sand will not freeze in winter.

Mr. Copeland. WILLIAM R. COPELAND, ASSOC. AM. SOC. C. E. (by letter).—In his interesting paper on the water filters of Bremen, Mr. Goetze has pointed out, that, if sand filters are used to purify clay-bearing waters, the effluents will not be clear unless the raw water applied has been clarified partially by passing it through preliminary filters.

The City of Philadelphia is carrying on a series of experiments with preliminary filtration, and it may be interesting to note the effect which such a method of treatment produced on the volumes of water that the secondary or slow sand filters purified in a few instances between scrapings.

As the water of the Schuylkill River often carried large quantities of clay, etc., the raw river water was passed through beds of filtering material at rates which varied from 60 000 000 to 110 000 000 galls. per

acre per day during these experiments. Even at these high rates, the preliminary filters took from 50 to 70% of the foreign matter out of the water. The solids in suspension were not always reduced to 50 parts per million, however, which is about the limit that slow sand filters can handle in a satisfactory manner. Mr. Copeland.

The data in Table 11 have been selected, therefore, because they give a good idea of the effect which different quantities of suspended matter in the applied water had on the amounts of water filtered between scrapings.

TABLE 11.

Slow sand filter No.	Turbidity of the applied water, in parts per million.	Millions of gallons filtered between scrapings.	Nominal rate of filtration, in millions of gallons per acre per day.	Maximum loss of head, in feet.	Effective size of sand, in millimeters.
7	56	236	6	3.6	0.41
7	5	849	6	3.0	0.41
B	627	20	6	5.7	0.42
B	28	400	6	5.0	0.42
B	56	177	10	6.1	0.42
B	12	963	10	6.3	0.42

The water applied to these filters was "pre-filtered" Schuylkill River water. Filter No. 7 and Filter B were composed of beds of sand supported by layers of gravel, and the filters were run in a careful and scientific manner. The figures in Table 11 show that, when the applied water contained more than 55 parts of suspended matter per million, the filters were only able to purify about 200 000 000 galls. per acre between scrapings; but when the suspended matter in the applied water was reduced to 12 parts or less per million, the filters purified more than 800 000 000 galls. per acre between scrapings.

As the preliminary filters remove the coarser particles of suspended matter from the water, it follows that the particles of foreign matter in the effluent are very small. Such very small objects find their way easily between the grains of sand, and do not clog the filter to any considerable degree. For that reason the loss of head increases at a slow and irregular rate, advancing on one day and falling back the next. In order to show how differently a filter will act, in this respect, the losses of head, for Filter No. 7, at 12 o'clock noon, of each day of the periods referred to in Table No. 11, are plotted in Fig. 1.

By an examination of Fig. 1 it will be seen, in the first place, that the loss of head increased more slowly, and, in the second place, that it increased much more irregularly, when the applied water contained 5 than when the applied water contained 56 parts of suspended matter per million gallons of water.

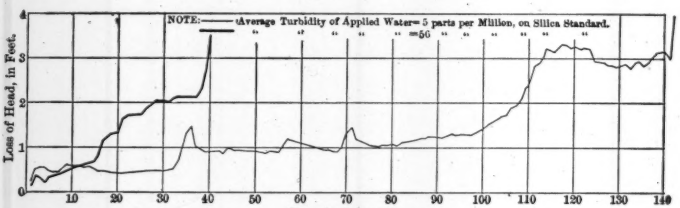
Mr. Copeland. Moreover, the filter would probably have run for several days longer in the first case if pre-filtered water had been applied after the 142d day. On the morning of the 143d day, however, raw river water was turned on the filter, and within two hours No. 7 went out of service.

The lessons taught by these figures and the diagram are:

First, that when a river water has been well clarified by passing it through efficient preliminary filters, slow sand filters will complete the purification at rates of 6 000 000 galls. per acre per day, or double the usual rate.

Second, that slow sand filters will treat very large volumes of a well-clarified water before the maximum loss of head develops.

Third, that the process of preliminary treatment must be very thorough in order to secure the most economical results.



RESULTS FROM FILTER NO. 7.

FIG. 1.

Mr. Le Conte. L. J. LE CONTE, M. AM. SOC. C. E. (by letter).—The writer is greatly interested in this most instructive paper. The happy results obtained are increased in value by reason of the fact that they are the results of well-tried experience, and are therefore entirely free from mere speculation. It seems to the writer that the author has certainly taken an important forward step in the art of sand filtration, and has opened up an entirely new field for future development, which can hardly be overestimated, either in its extent or in the importance of the results. Every true student of sand filtration will hail with delight any reliable effort made in the art which will tend toward the avoidance of the use of chemicals for precipitation. The results at Bremen with water from the Weser—which seems to be quite muddy during storms—show conclusively that in that case the use of chemicals was entirely unnecessary. It remains to be seen to what extent this very important principle can be pushed with economy and safety. Now that the facts have been well established by experience, it remains for someone to come forward and explain.

After reading over the results one can hardly escape the conclusion that, after all, the good work done is simply one of natural avidity.

Mr. Le Conte.

During ordinary stages of the river, the raw water contains no more food than the colloidal film—covering the upper surface of the filter—can get away with conveniently. During flood stages, the raw water is more heavily charged with food products than the colloidal film can conveniently get away with, and, consequently, the film germs get more food than they actually need, and, therefore, allow it to pass through with the filtrate.

Not so with the second filter, which is fed with the filtrate from the first filter. Here the film is fully developed and ripened with raw water before it is put into service as a second filter, and, subsequently, it is fed with the first filtrate, which is necessarily lacking in food products. Hence the film germs are naturally fierce and hungry, and consequently attend to their duties with much avidity, and the result is a high-grade second filtrate. This would seem to be a reasonable way of looking at it; at all events, it is offered as a possible explanation of this most important phenomenon.

These interesting results, among other things, hold out pleasing hopes for the turbid water supplies from the southern rivers, but the news seems to be almost too good to believe fully that such a simple procedure could possibly overcome the great difficulties contended with at Cincinnati, St. Louis and New Orleans. The writer has no doubt that double filtration would certainly palliate, very largely, the troubles, but, nevertheless, he thinks that possibly triple filtration might be found to be necessary.

The author's automatic regulators for controlling filtration are very ingenious in design, and are worthy of high praise for their simplicity in detail.

The operating expense of double filtration, given by the author as $1.52 per million gallons, is most remarkable for economy, as it means, approximately, $3.04 per million gallons in this country, which is far less than one would naturally expect.

The Society is greatly indebted to the author for this most valuable paper.

Mr. Maignen.

J. P. A. MAIGNEN, ASSOC. AM. SOC. C. E.—This paper is very interesting, not only because it contains a practically new idea in the management of slow sand filter beds, that of filtering a second time the filtered water when it does not come up to the desired standard, but also because it deals with a number of questions which may be discussed with advantage, for instance:

The best test to apply to filters;

The influence of bacteria in the process of water purification;

The "ripening" of filters;

Whether or not all the dirty sand should be scraped;

The speed of filtration;

The meaning of the words, "preliminary" and "final" filters, etc.

Mr. Maignen. The discussion of this paper will, no doubt, bring out much useful information and reveal some of the progress made in the art of filtration during the last few years.

The author suggests that "the bacteriological examination of every filter should be made daily, if possible." In great cities like Philadelphia, New York or Washington this is possible, but in small towns, where the resources are limited, it is not practicable. The speaker, however, thinks that all public water supplies, large or small, should be examined bacteriologically at least once a month and preferably once a week, and he suggests that periodical qualitative analyses of the raw water and of the filtered water (as it enters the distribution mains) would be productive of more good than the daily quantitative test of each individual filter.

To this should be added some simple chemical tests for the detection of organic matter and chlorine, which would indicate immediately any extraordinary pollution of the water supply, upon the ascertainment of which, public authorities should seek and abate the nuisance.

The qualitative analyses will show the kinds of bacteria which pass through the filters, and establish whether or not the purification is complete. If it be found incomplete, those in charge of the works will be induced to look out for improvements. At present it seems to be granted that a certain number of *Colon Bacilli* may pass through the filters without impairing the potability of the water. If the speaker were responsible for the management of a public water supply, he would not rest until the system or systems of filtration used were so perfect that they would give a final filtrate entirely free from *Colon Bacilli*.

The speaker agrees with the author as to the advantage of a physical test, or turbidity observation, because it gives "immediate results." While the turbidity standards recommended by Mr. Hazen, Mr. Whipple and Mr. Goetze may be satisfactory for very turbid applied water, in the opinion of the speaker, they are not as good as the simple tube for filtered water, which is, or ought to be, always practically free from appreciable turbidity.

The tube used for this purpose at Bremen is a practicable device which an ordinary filter attendant can easily handle. The color of the water in that depth is a very good indication of the operation of the filters. In London, the chemists, appointed by the Government to test the quality of the water supplied to the metropolis by the different water companies, used to publish a weekly report stating that the filtered water, as seen through a 2-ft. tube, was yellow, green or brown. These different shades indicate varying proportions of suspended matter in the water.

In Paris the speaker used a 3-ft. tube for the rapid and frequent

examination of each filter, and a 15-ft. tube for examining, once a day, the general filtrate from the pure-water well. In one case he had to deal with a filter plant having a very great number of units (more than 2 000) divided in sixteen sectional collectors. When all the units were working normally, the water in the tubes, either small or large, did not obscure the light in the slightest degree, the blue sky appeared as blue and the white clouds as white as if no water had been in the tubes; but as soon as a single unit went wrong the water in the short tube lost its brilliancy and that in the long tube appeared slightly yellow. When several units were defective the color was more intense, and turned to green, and, as the defective units increased in number, the water became brown, the raw water, of course, obscured the light entirely and appeared black. As soon as all the defects were corrected the filtrate became again absolutely colorless. The speaker would recommend the use of a 2-ft. or 3-ft. tube for checking the daily work of each filter. Mr. Maignen.

The author affirms that the "sand, *per se*," is not a true filtering medium; that it is "the slimy coating (*Schlammdecke*) which is deposited upon and between the grains of the upper layer of sand" that does the work. He speaks of the "favorable interactions of bacteria and algæ," and of "organisms consuming and producing oxygen." He states, further, that "a filter bed filled with clean sand is not an apparatus which could be called a filter, * * * only by the so-called *Schlammdecke* is there formed the filter which is suitable to retain the bacteria;" and again, that the sand layer is used as a biological filter.

The speaker is of opinion that the author attributes too high a value to the *Schlammdecke*, or *Schmutzdecke*, or, in plain English, "mud blanket," and to olittle to the sand itself, and that the effects of the so-called biological action, so much dwelt upon during the last ten years, have been overrated.

Further, he believes that the surface of a sand bed is a hecatomb for the water bacteria, and not necessarily a breeding ground for them.

In open filters the water insects and infusoria may feed on the bacteria, fresh-water sponge and algæ, which grow by the favor of the light, act as a kind of preliminary filter, and may entangle some of the bacteria; but it is not likely that the bacteria themselves eat one another; they are retained like the rest of the suspended matter, and contribute physically to the formation of the complex substance which is called "mud."

In covered filters, where the sunlight does not penetrate, there is much less animal and vegetable growth, yet the speaker is not aware that the filtrate of covered beds is inferior to that of open beds.

The speaker calls attention to his observations on "ripening" filters,* in his discussion on "The Filtration Works of the East

* *Transactions*, Am. Soc. C. E., Vol. L, p. 447.

Mr. Maignen.

Jersey Water Company, at Little Falls, New Jersey." There he showed that sand or granular-bed filters could be rendered "ripe" by filtering practically sterile water during a certain time, say one month. They became "ripe," that is, capable of giving an almost sterile filtrate without the formation of any kind of "mud blanket," and they continued to give perfect results for months afterward with applied water very rich in bacteria.

The author recommends that "the coherence of the bacterial mass beneath the removed layer must not be disturbed"; he advocates the removal of only the "most dirty layer" and not the whole of the "slimy coating." There are filter plants in this country in which all the dirty sand is not removed, and others where it is entirely removed, and, as far as the speaker knows, it is the latter filters which give the best effluent; therefore, he cannot agree with the author on this point.

The author acknowledges that a "ripe" filter may deliver a poor filtrate during floods. According to the biological theory, when a filter is old it is "ripest," and should, then, if the theory were strictly correct, do its best work when it is oldest, yet at such times, if the applied water is roily, the filtrate becomes poor, and, therefore, an explanation, outside of the biological theory, must be sought.

The speaker ventures to suggest the following theory for the breakdown of "ripe" filters, when the applied water becomes bad. In the act of filtration, the water in its downward course gradually creates microscopical fissures, passages or channels, after the fashion of underground waters which flow long distances in fissures. As long as the applied water is comparatively clear there is no disturbance in these filtering channels, the particles of suspended matter are comparatively large and light, and are easily retained on the surface, but when road wash and finely divided clay are brought down by heavy rains a new factor intervenes, the fine particles of clay, microscopical stone detritus and other amorphous matters are carried with the water through the above-described channels, and it is believed that by their weight and number, as well as by their irregular form, they have a tendency to increase the erosive action of the flowing water and actually enlarge the channels through which they pass. This fact is particularly noticeable by the discoloration of the sand in some parts of the filters, and particularly along the walls and piers.

Something has been said about the unsatisfactory operation of sand filters in winter. May there not be some such erosive action produced by the cold water, which, of course, is not as fluid or filtrable as the warmer water of summer?

The author recommends that the water should be refiltered when the first filtrate is poor, particularly after refilling and in time of flood. By doing so he applies on a large scale what is done in the

laboratory when precipitates are separated from fluids by filtration through filter paper. The first filtrate is usually cloudy because there are pores in the paper larger than others, when these become masked by a portion of the precipitate the liquid comes out clear, an artificial filtering membrane has been formed and the heterogeneous surface has been rendered homogeneous. This is essentially a physical process. The author's process differs from this only in the fact that, instead of passing the first filtrate through the original filter, he passes it through another filter of like character. Mr. Maignen.

The author compares the action of slow sand filters to Nature's process, and says:

"In filters which are carefully provided with a sand layer of even grain, and through which the flow of water is regulated according to the best means known to the art, a process of cleaning (purification) takes place analogous to that which in Nature produces the highly praised spring water."

As a matter of fact, sand, in Nature, has not much to do with the purification of water. It is known that when a cesspool is within easy distance of a well, and when the soil in which both are located is made up of "sand of even grain" there is "infiltration," or slow filtration, of the poisonous matters from the cesspool into the well. The same kind of unsatisfactory result occurs with what is known as "surface springs," whether the surrounding soil is sandy or otherwise; when the rain falls the so-called spring water becomes cloudy.

In Nature, the water found in sand-bearing strata is not dirty when it gets there originally. The water of Nature's springs generally comes from distant forest lands where the mossy growth acts as a preliminary filter, or it may come from slowly melting snow, in which case the loamy soil also acts as a preliminary filter. In any case, the mud does not get into the sand, hence it is that the so-called natural sand filters, which give up their water in the shape of springs or wells, never get clogged.

Many engineers have been tempted by the idea that the gravel soils which are on a level with the river banks could be converted into natural sand filters by being drained into so-called filtering galleries, but much disappointment has been met with by those bold enough to try. For a few years, perhaps, very fine water is thus obtained, but a time comes when the water originally stored in the sand strata is practically exhausted; the fresh water which flows from the river into the gravel carries with it mud which accumulates at the points of contact between the liquid and solid elements, and the mud itself blocks the "natural" filters as it blocks the embankments of canals or the surface of artificial sand filters. If any water then penetrates the gravel, it does so, not through a great number of small filtering passages, but through large fissures or "rat holes."

For instance, at Nancy, in France, filtering galleries were estab-

Mr. Maignen. lished in sandy soil alongside the River Meuse; for a time, plenty of water found its way into the galleries, and it was of very satisfactory quality, but, after a certain period, the mud clogged the sand nearest to the river, and the sandy soil surrounding the galleries became dry. The galleries themselves continued to give water, but it was unfiltered, it came direct from the river through large fissures, and caused considerable sickness in the town.

It should be remembered that the art of filtration is not limited to the passage of water through a porous body, it extends also, and we may say, above all, to the keeping of the filtering surface clean. The prize will be for those who will devise the best and most economical means of disposing of the mud.

So-called "natural" sand filters are not easy to clean, therefore they may be dismissed from consideration at present.

If it is desired that sand filters work as in Nature, care should be taken that the water sent on them be as free from suspended matter as it is in Nature. Sand makes a very good bacteriological filter, the bacteria do not find sustenance in it, and if the applied water, as stated by the author, is fairly good all the time, the filtrate is always well nigh perfect; therefore, it may be laid down as a principle that all water supplies intended for sand filtration should receive a preliminary treatment.

What is done at Bremen cannot be correctly called "preliminary" filtration and "final" filtration; it would be more accurate to call it "first" and "second" filtration.

The speed of filtration recommended by the author, namely 1 600 000 to 2 000 000 galls. per acre per day for the "first" filtration and 2 600 000 to 5 000 000 galls. for the "second" filtration, may be practicable in places where the consumption is from 20 to 30 galls. per head per day, but it would be altogether out of the question in this country, where the *per capita* requirements vary from 60 to 200 galls., on account of the enormous expense of first installation.

Downward filtration through sand, coarse or fine, slow or rapid, cannot, in the speaker's opinion, be recommended for "preliminary" filtration. It must always be an expensive operation, because the voids between the grains of sand are necessarily small, and they become very quickly closed up, the quantity of sand necessary to retain a certain quantity of mud is very considerable, and the handling of the dirty sand, whether it be washed *in situ*, as in mechanical filters, or removed, as in slow sand filters, forcibly necessitates much power, labor and wash-water. Whatever qualities as a "final" system sand filtration may have, it cannot be considered an economical process of preliminary filtration.

Upward filtration through sand, also, cannot be thought of for preliminary filtration, for the same reason, that too much material has

to be handled for a given quantity of mud, and also because the upward flow of water through sand transforms the filter bed into quicksands and surface springs. Mr. Magnen.

A true system of economical preliminary filtration has been inaugurated quite recently in Philadelphia, at the Lower Roxborough filter plant. It will no doubt be brought to the attention of the Society in detail some day, but, on this occasion, it may be briefly described:

The filtration is upward; the materials used are, first, a layer of slag, in pieces varying in size from 3 ins. to ¾ in., and second, a layer of sponge. There is practically no loss of head, the effluent rising within 1 or 2 ins. of the level of the influent, and the operation goes on for three months or more without any attention or labor of any kind. The slag is cleaned by downward, reverse flushing, and the sponges are washed in laundry machines.

There is an old saying in Germany to the effect that when dirty water has run over nine stones it becomes clear, alluding, no doubt, to the behaviour of wash-water or sewer-water in village brooklets. This is well illustrated in the Philadelphia preliminary filter, which has a bacterial efficiency varying from 80 to 95%, with a speed of filtration of 48 000 000 to 60 000 000 galls. per acre.

In this preliminary filter there is plenty of room for the mud, and the quantity of material to be handled, for a given quantity of mud, is reduced to a minimum, hence its economical operation.

The object of the Bremen system of intermittent double filtration is to insure at all times a good filtrate; the object of the preliminary filtration, as carried on in Philadelphia, ought to be, in the speaker's opinion, threefold:

First, to insure a good filtrate at all times;

Second, to reduce the cost of first installation by allowing the final filters to work at higher rates, say 6 000 000 galls. or more per acre per day, and

Third, to cut down the cost of operation of the "final" filters by reducing the number of sand-cleaning operations.

GEORGE W. FULLER, ASSOC. M. AM. SOC. C. E.—This interesting paper makes it plain that the City of Bremen has a filter plant which is well built and well operated. That view is certainly confirmed by those who have had an opportunity to visit the works and examine the results of the many studies made by the author, and of which those presented in this paper are representative. Mr. George W. Fuller.

There is one feature, not mentioned by the author, which is of importance in recording the accomplishments of a filter plant which has been in service for a long term of years, and that is the effect which has been produced, upon the people who drink that water, with reference to the prevalence of water-borne diseases, such as typhoid fever.

Mr. George W. Fuller.

As the speaker remembers the statistics, the annual death rate from typhoid fever in the City of Bremen is only about ten per hundred thousand of population. That, it will be recalled, is less than one-third of the proportional number of deaths from this disease in the average American city, and about one-half of that in those cities in this country which are considered to be provided with very good water.

There are, of course, other factors besides the water supply which are related to the proposition of low death rates from typhoid fever, but it seems that, in connection with this paper, it is very desirable to bear in mind the actual accomplishment of the filters which is aimed at; that is, to give the people a water which is not only pleasing in its appearance, but one which is thoroughly safe to use for drinking purposes.

In the management of the Bremen filters, there is a feature which is worthy of considerable study by the water-works engineers of this country. Even with such devices as the by-passes mentioned by Mr. Hazen, where opportunities are available for getting into trouble, it is significant that those troubles have not come. Great influence, in the opinion of the speaker, ought to be exerted by engineers in this country toward bringing filter operations to a higher plane of excellence. While many filter plants in this country, both of the sand-filter and the mechanical-filter types, are well operated, there are many which are not so well operated. It is believed that this is a very important factor to be borne in mind, in connection with the development of water purification in America.

Why the Bremen filters do not produce a better effluent when for short periods a moderate turbidity appears in the river water, is a question which requires considerable study, and is difficult to answer.

With filters of this type, operated at several places in the United States, especially on the Ohio River and on the Mississippi River at New Orleans, they have been able, with a turbidity greater even than that encountered at Bremen, to produce a good filtered water for quite a number of days after muddy water appeared. Frequently, as many as ten days have gone by, with very turbid water going on the filters, and still the filtrate was satisfactory in appearance and excellent, bacteriologically.

These differences in experiences are doubtless due to differences in conditions which are difficult to gauge. How far the character of the organic matter which forms films upon the sand grains (of which so much is said in Germany, but which in this country is not considered so much a factor) may explain these particular results, it is hard to say. It seems to the speaker that the shallower sand layers, which they have used at Bremen and other places in Europe, may be a factor of some consequence.

Mr. George W. Fuller.

Speaking generally of double filtration, it appears that there are many waters in this country where there may be opportunities for the advantageous application of such a method. Some of them are in connection with those waters which become turbid for short periods, but not sufficiently turbid to render necessary large investments for settling basins or coagulating basins.

Others refer to waters which have bad tastes and odors, due to vegetable growths, such as that which has been studied for many years at Springfield, Mass. The studies of the water of the Ludlow reservoir are along lines different from those which have been usually studied elsewhere. They refer to the removal of tastes and odors produced by *Anabæna*, a vegetable growth which seems to have oil sacs contained in it. Upon dying, this oil, which is very pungent and penetrating, is liberated. A small quantity of it will affect a large volume of water.

Double filtration is useful, in connection with such waters, in facilitating oxidation, and can produce a good effluent; whereas, with single filtration, the result at times is decidedly unsatisfactory.

In general, double filtration is something of which a great deal more will be heard in the future.

Mr. Bryan.

WILLIAM B. BRYAN,* M. INST. C. E. (by letter).—The writer, when appointed Chief Engineer of the East London Water-Works Company, found two systems of sand-washing in use: One was merely a brick-work tank, with end walls only, into which the dirty sand was wheeled. The workmen cleansed the sand with a jet from a hose pipe. The cleansing was very effectual, but the loss of sand and the consumption of water were very great. This system is still in use in some water-works in England.

In the second system, the dirty sand was thrown into a cast-iron tank with a perforated bottom, and the water, under pressure, flowed upward and agitated the sand. The dirty water flowed away into the drains and catch-pits provided for the purpose.

Both these systems, although doing their work effectually, were costly in operation. In 1882 the writer visited the Birmingham Water-Works, where he saw sand being elevated out of one tank into another, only 3 or 4 ft. high, by three 1½-in. pipes laid on the slope, and three ordinary jets of water, under pressure, directed into these pipes (Fig. 2). The sand was elevated with a very fair amount of success. This was not intended as a washing apparatus. Some time later, the writer had an ejector made, of a very large size, like the sketch in Fig. 3, but it was intended to work the apparatus in pairs. The top was to be closed by a block of wood, about 10 ins. in diameter, after the ejector was filled with dirty

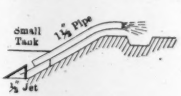


FIG. 2.

* Engineer, East London Water-Works Company.

Mr. Bryan. sand. This was merely an experiment, and was never used in actual practice on account of its enormous weight and size. When a few bar row loads of sand and gravel were thrown into the ejector, and water under pressure was turned into it, the gravel and sand were delivered at a distance of about 700 ft. and to a height of about 11 or 12 ft. The experiment was very successful, but it showed at once that for practical work something which would be continuous in operation must be obtained, and, after a number of experiments in 1884, 1885 and 1886, by a gradual process of evolution, and after examining all the various kinds of apparatus used in South Wales for washing coal, the present system was introduced.

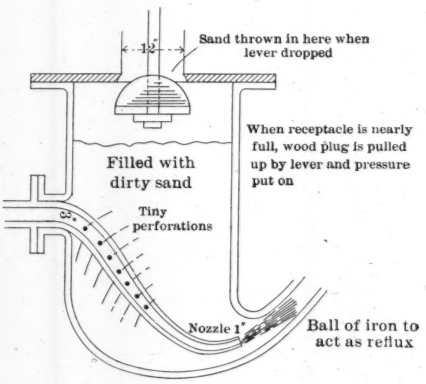


FIG. 3.

At first, it was proposed to use several ejectors, one after the other, so as to do away with machine washing, but it was found that machine washing, after taking everything into consideration, was better than any other; and probably, in a great measure, this is due to the shape of the East London filter beds and the available space in the center enabling the rotary machine to be swung around on a little railway along the edge of the circular, covered, pure-water basin. The castings for the ejectors were made by Messrs. Hunter, of Bow, for the East London Water-Works Company, and, a year or two later, Mr. Hunter, in conjunction with Mr. Frazer and Mr. Goodman, of the Grand Junction Water-Works, took out a patent for sand-washing by ejectors, but, unfortunately for them, they had been anticipated by several years by the writer. As will be seen from Figs. 4 and 5, and the photographs on Plate XV, the dirty sand, instead of having to be wheeled a long distance out of the beds, as formerly, is merely wheeled into a cast-iron hopper, the water is turned on, under pressure, and left on until the whole of the dirty sand in the bed had been lifted out. It passes into one end of the rotary machine, and is also washed in transit. A little hydraulic engine, fed by pure filtered water, turns the arm inside the longitudinal cylinder, and the exhaust water flows into the hopper at the opposite end from the dirty sand, consequently, there is always a current of water meeting the sand passing through the washer. The dirty sand overflows at one end, and, as the sand gets cleaner, it

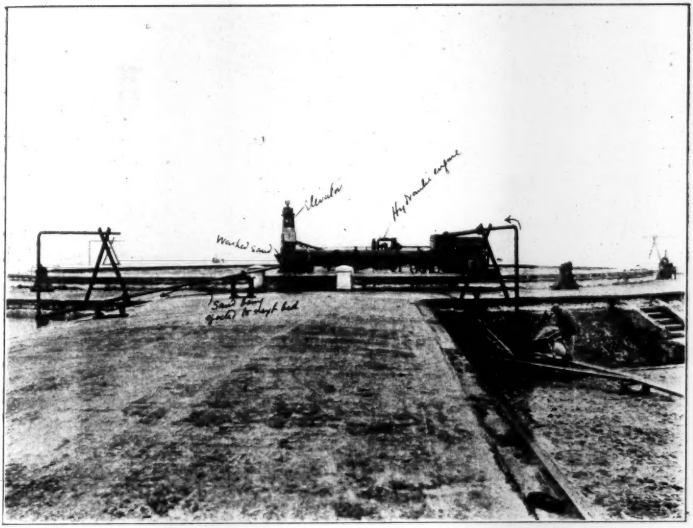


FIG. 1.—SAND-WASHING APPARATUS AT THE EAST LONDON WATER-WORKS.

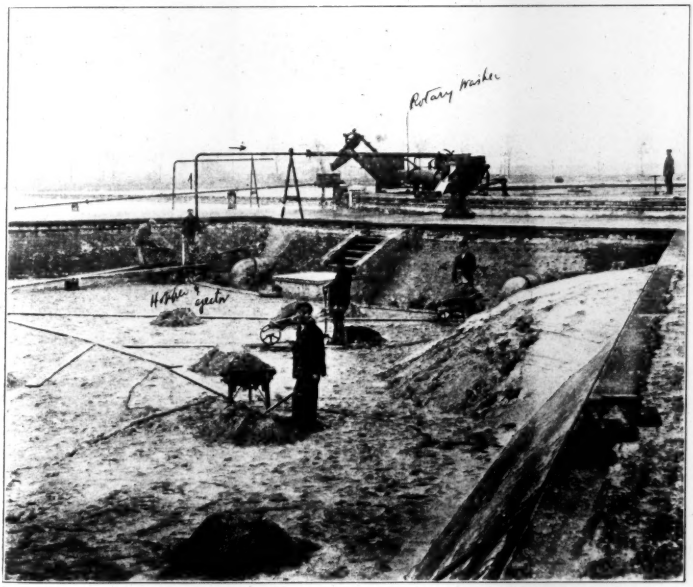


FIG. 2.—SAND-WASHING APPARATUS AT THE EAST LONDON WATER-WORKS.

meets the cleaner water. The elevator drops it into a small iron receptacle and a ⅜-in jet of water throws the clean sand into an adjoining bed, the latter operation involving no loss of water whatever. In the original experiments an unforeseen difficulty was met in connection with the suction pipe. This pipe was made somewhat coned, as in the sketch, Fig. 6, with the widest end of the cone dipping into the dirty sand, and the flow of water was so smooth that the organic matter in the sand formed a slime on the surface to a considerable distance, and

Mr. Bryan.

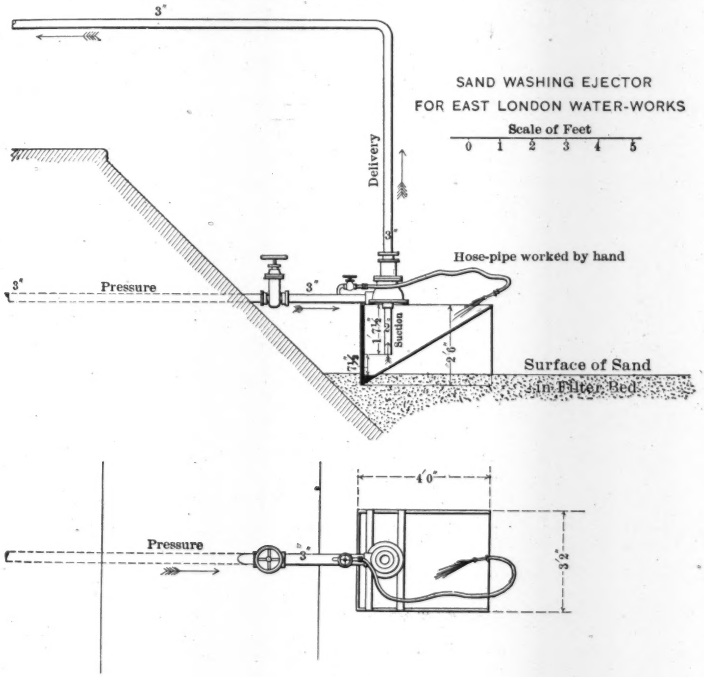
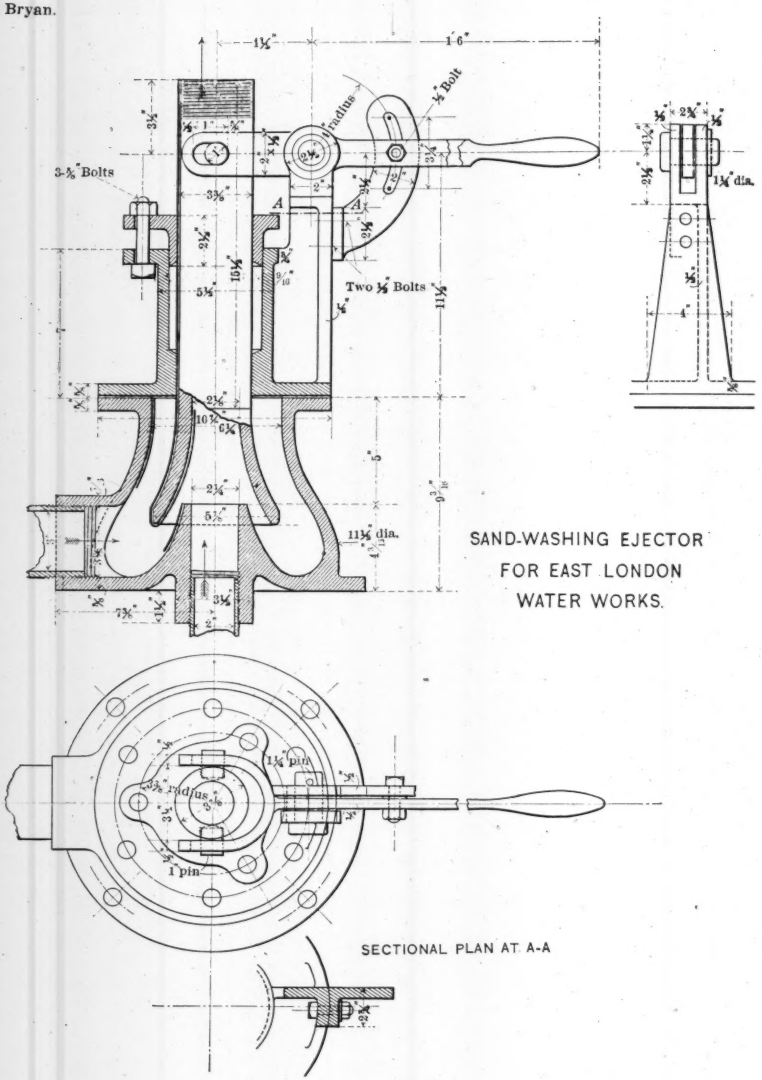


FIG. 4.

prevented its efficient operation. Therefore, the suction pipe was taken off and a straight one put in with the water entering a T-piece at the bottom with holes drilled through so that it should enter four ways at once and the water jostle itself in all directions in passing up the pipe. No trouble has been experienced during the last seventeen years.

Shortly after the great epidemic of cholera at Hamburg, the engineers of the Hamburg Municipality paid a number of visits to

Mr. Bryan.



SAND-WASHING EJECTOR FOR EAST LONDON WATER WORKS.

FIG. 5.

the East London Water-Works, and were very much impressed with the ejecting apparatus. As the writer was known personally to Messrs. Körting, of Hanover, who were great makers of similar apparatus, he suggested that that firm would (when they had designed and constructed their new filters), no doubt, turn out as good an ejector as that being manufactured for the East London Water-Works, and this has been done; and the Chief Engineer of the Hamburg Municipality has always given the East London Water-Works Company the credit for being the pioneers in ejecting dirty sand from filter beds. Mr. Bryan.

FIG. 6.

RUDOLPH HERING, M. AM. SOC. C. E. (by letter). The writer has been requested to close the discussion on Mr. Goetze's paper, having urged its presentation to the Society. He is pleased to observe that the paper has been found both interesting and instructive to nearly all the discussors, and that it has elicited some valuable comments in relation to the application of the described system to the conditions in America. Mr. Hering.

Mr. Kiersted and others very properly emphasize the caution that very turbid waters, such as those in the Southern States and those of the Middle West, could not be successfully treated alone by the system of double filtration, because a prior settlement of the water is essential in such case for economical reasons.

There remain, however, as remarked by Mr. George W. Fuller, many waters in America where there may be opportunities for the advantageous application of double filtration; some, where the cost of settling or coagulating basins does not seem warranted on account of very short periods of turbidity, and some where bad tastes and odors cannot be satisfactorily removed by single filtration.

Mr. William B. Fuller's contention that the Bremen method of double filtration might be dangerous here, because in America slow sand filters must be made as nearly "fool-proof" as possible, is rather more a reflection upon us than upon the intrinsic merits of Mr. Goetze's design. Mr. Hazen reiterates an earlier statement of his, saying that with careless or vicious attendants the raw water of the Bremen filters can be run into the filtered-water pipes. Mr. Goetze has stated that such a case has never occurred, and could not occur with even fairly intelligent management. It seems to the writer that the operation of any modern filter, of whatever kind, requires careful and conscientious work. By "careless and vicious attendants" any one of them may be made ineffective, and, like the neces-

Mr. Hering. sity for capable attendants to operate high-class modern pumping machinery, where "careless or vicious attendants" might cause the loss of much capital, so does a similar fear not seem to justify opposition to the introduction of a higher class of intelligence in the operation of filters.

Mr. Knowles maintains that in America the introduction of double filtration means the necessity of installing twice the otherwise required filter area. This would be true for very turbid water. But it hardly seems true, when the application of the system is limited, as it should be in America, to the cases mentioned by Mr. George W. Fuller, and which correspond closely to the Bremen conditions.

Mr. Hazen believes the winter temperature of Bremen to be about the same as that in Pittsburg, and that covering filters in the climate of the latter city is to be regarded as a luxury rather than necessity. In a report of a commission on the Pittsburg case last winter, of which the writer was a member, the following table of winter temperatures convinced the entire commission that for at least last winter the filters if uncovered would have been inoperative, and it seems to the writer to have been a very prudent decision, not only in Pittsburg, but even in Philadelphia and Washington, to cover the projected filter beds:

MINIMUM TEMPERATURES AT PITTSBURG, PA.

Dec., 1903	28	days	below 32°
	14	"	" 20°
	4	"	" 10°
	0	"	" 0°
Jan., 1904	26	"	" 32°
	18	"	" 20°
	8	"	" 10°
	2	"	" 0°
Feb., 1904	27	"	" 32°
	18	"	" 20°
	14	"	" 10°
	2	"	" 0°

In Hamburg, Germany, the filters would have been covered by Mr. Meyer, the engineer, without question, as he told the writer, had the condition of the soil allowed of it.

Regarding the use of the ejector type of sand washer, which the author does not favor, American experience, on the contrary, rather favors it, with certain improvements suggested by Mr. W. B. Fuller and Mr. John H. Gregory, and it seems that this type has come to stay with us for a while, as it is effective, convenient and economical.

Mr. Maignen and Mr. Copeland give some interesting statements regarding certain experiments at Philadelphia on preliminary filtration

of another character, indicating that slow filters when receiving water that has been well clarified by a thorough preliminary filtration can double their rate, and indicating that hereby much larger volumes of water than with single filtration can be treated, before the loss of head puts the filters out of service. The method, however, is radically different from that in use in Bremen, and further experience must be gained before a verdict as to the relative merits and applications of the two systems can be given. Mr. Hering.

The writer believes that Mr. Le Conte has a rather too rosy expectation regarding the effect of double filtration on such very turbid waters as those in the South and Middle West. While repeated filtration might reduce, and, for short periods, sometimes completely remove, the turbidity, it is pretty certain that in the waters mentioned this could be done only at a much greater expense than when a system of coagulation is substituted, as projected in a number of plants now in process of construction in that territory.

It is very gratifying to receive Mr. Bryan's discussion, as it gives an interesting account of the evolution of the sand-washing device in England and in Hamburg.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 983.

LATERAL EARTH PRESSURES AND RELATED PHENOMENA.*

By E. P. Goodrich, Jun. Am. Soc. C. E.

with Discussion by Messrs. M. F. Bonzano, V. H. Hewes, H. F. Dunham, Robert Brewster Stanton, Richard Lamb and E. P. Goodrich.

Introduction.

While theories concerning the action of granular masses, as to planes of rupture and of friction, and as to lateral pressures, etc., are fairly numerous, they are not at all concordant in the results they produce, and these results are not in accord with the few experiments thus far made public, or with common engineering practice in the design of structures having to do with such granular substances. It is hoped that the experiments herein described will throw some light upon the subject, aid designers in getting nearer to the requirements in actual cases, serve to correct some errors in the several theories, and form a nucleus around which to build practicable working rules or perhaps a theory which will indeed agree with fact.

The order of presentation of the following material is almost the opposite of that in which it was worked out.

* Presented at the meeting of April 20th, 1904.

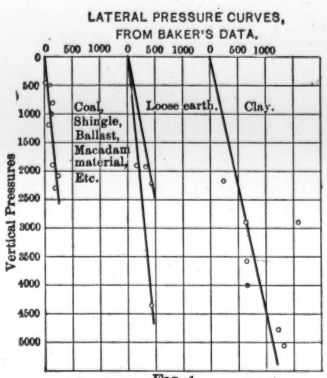


FIG. 1

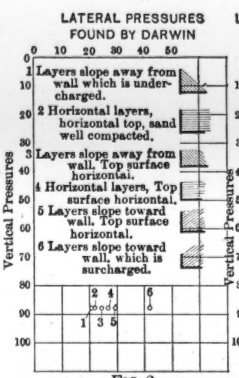


FIG. 2

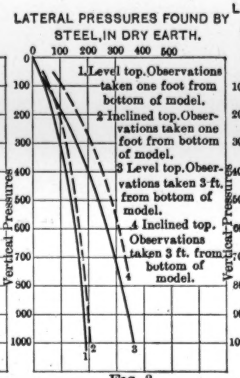


FIG. 3

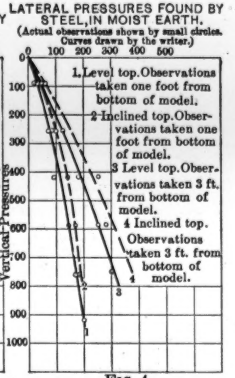


FIG. 4

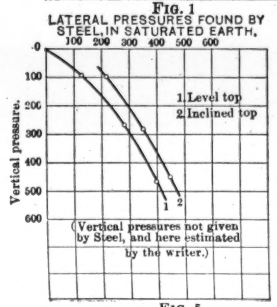


FIG. 5

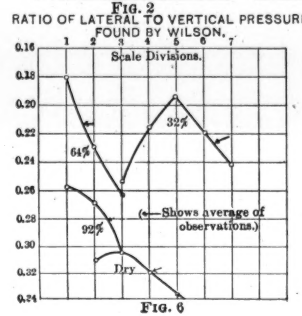


FIG. 6

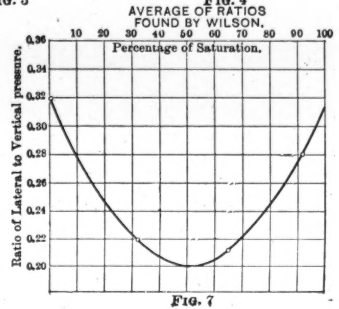


FIG. 7

Work of Other Observers.

An analysis of the data given by Sir Benjamin Baker, in his paper,* "The Actual Lateral Pressure of Earthwork," warrants perhaps one deduction, *viz.*, that the coarser the materials, the less the lateral pressure. The curves of Fig. 1 show that, for coal, shingle, ballast and macadam material, the results are fairly concordant, and that the lateral pressure for such materials is about one-tenth of the vertical pressure, and varies uniformly with it. It appears that the lateral pressure for clay runs often to one-fifth of the vertical pressure, while in loose earth it varies between one-tenth and one-fifth.

The work of G. H. Darwin, described in his paper,† "On the Horizontal Thrust of a Mass of Sand," is of little practical value because of the small size of his model, but several of his conclusions are interesting. He says, "the coefficient of maximum internal friction is probably very different in different parts of a mass of sand," and it is not equal at any point to that of the talus of greatest possible slope, and "is a function of the pressure," and, also, "of the pressure and shaking to which at some previous period the mass of sand has been subjected." Fig. 2 shows the range of his experiments, plotted to a scale for comparison with others.

A. A. Steel, in his article,‡ "Experiments in Earth Pressures against Retaining Walls," concludes, in part, from his work: That the lateral "pressure is not equal to a constant times the head" (vertical pressure); that the surface slope has some slight effect upon the lateral pressure; and that "the nearness of the solid bottom" (of his model) § "has an unknown but large effect on the lateral pressure." His last conclusion is, really, that the compressibility of the earth, combined with its arching effect, affects largely the lateral pressure. The results of his experiments, in terms of vertical and lateral pressures, are shown in Figs. 3, 4 and 5. For dry and moist earth, he finds that the lateral pressure is from one-fifth to one-third of the vertical pressure, and, in saturated materials, is practically equal to it.

The excessively large and the almost minute pressures used by George Wilson, described in his paper,‖ "Some Experiments on Con-

* *Minutes of Proceedings*, Inst. C. E., Vol. LXV, p. 140.
† *Minutes of Proceedings*, Inst. C. E., Vol. LXXI, p. 350.
‡ *Engineering News*, Oct. 19th, 1899, Vol. XLII, p. 261.
§ The words in parentheses are by the writer.
‖ *Minutes of Proceedings*, Inst. C. E., Vol. CXLIX, p. 208.

jugate Pressures in Fine Sand and Their Variation with the Presence of Water," make his work of little value except for comparison. He proves that the tangent of the internal angle of friction, at pressures up to 100 tons, varies with the amount of moisture present. Also, by another series, that, even at depths of a few inches, the moisture present has considerable effect upon the coefficient of friction between sand and metal, sand and glass, and sand and wood. The results of his experiments, replotted for comparison with others, are shown in Figs. 6 and 7. He finds that, with differing amounts of moisture, the lateral pressure at great depths (approximating 1 000 ft.), varies from one-fifth to one-third of the vertical pressure, being—

"Greatest when the sand is dry, or when it is saturated with water; that it diminishes to a minimum between these limits and then increases again; and that the value of this decrement for any particular percentage diminishes as the pressure increases."

Experiments with Model.

One series of observations made by the writer was upon a model which could contain a mass of earth 3 ft. x 3 ft. and 6 ft. deep. (See Fig. 8.) Stiff corner posts were bolted together to make a frame, from which a box could be made by the insertion of removable side boards 3 ft. long and 1 ft. wide. On one side of the model the boards were fitted with especial care, and paper was placed over all edges so that the particles of earth could not enter the joints and affect the results. About 3 ins. above the center line of the lowest board upon this side, a rod, secured to the vertical posts, was placed so that two bent levers with fulcrums on the rod would each bear with a short arm against the center of the board near its two ends, and so that a long arm would extend horizon-

THREE-FOOT EARTH-PRESSURE MODEL.

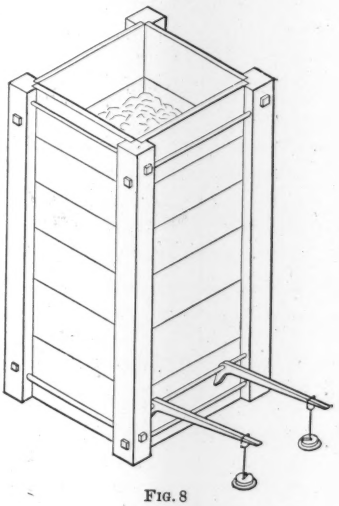


Fig. 8

tally from the model. A scale-pan was arranged to be hung on each horizontal arm, and weights of known values were cast for the pans. Thus, by moving the pans along the horizontal arms, varying pressures of known amounts could be obtained against the board, and, consequently, the lateral pressure of the earth inside the model could be equilibrated. In conducting an experiment, common, slightly moist, bank sand was first thrown into the model to the desired depth. Long, tapering, metal wedges were then inserted between the frame and the lowest movable side-board, and weights were gradually applied to the scale-pans and the latter moved along the lever arms until the wedges dropped by their own weight. The uniformity of the results depended upon the nicety and exact equality with which the wedges were inserted, and this depended entirely upon the skill of the observer. After a little practice, enough skill was acquired so that the results of the observations made on separate days did not differ more than could be attributed to other sources of variation, such as the manner of depositing the sand in the model, its varying humidity, etc. Furthermore, two consecutive series of observations by any individual observer did not vary more than 10%, including all sources of error.

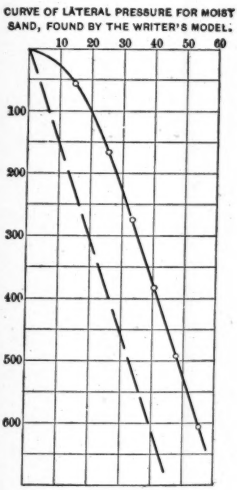


FIG. 9

Fig. 9 gives the averages of all results obtained with the apparatus shown in Fig. 8. In this experiment the lateral pressure is approximately one-fifteenth of the vertical pressure plus 15 lbs., thus being less than one-fifth of the vertical pressure, except near the surface, and diminishing with increased vertical pressures. The curvature of the upper portion of the figure shows the same departure from uniform variation found by Steel (see especially Figs. 3 and 5). The 15-lb. constant would immediately suggest a uniform friction increment or arch-action increment, and, doubtless, there were such increments, because of the small size and of the manner of construction of the model. The small ratio of lateral to vertical pressure also indicates such conditions, and the results, therefore, are believed to be of little value. It is possible that some

board above the lowest one would have given relatively larger results, similar to the ones found by Steel.

Experiments with Retaining Wall.

Another experiment was tried, in connection with the filling behind a line of sheet-piling driven to form the face of a retaining wall. A section of the piling was selected which was fairly straight and had been evenly driven. A box, 6 x 1 x 15 ft., inside measurement (see Fig. 10), was constructed without a top and with the sheet-piling as one of the large sides. This brought the other large side of the box broadside to the fill. This side consisted of 3 x 10-in. yellow pine plank, 15 ft. long, and was, therefore, of such material and dimensions as to permit of appreciable deflection by the lateral pressure of the fill. The box was well braced around the ends, and the top was left open, so that a measuring instrument could be dropped down inside of the box to ascertain the distance between the sheet-piling and the deflected side at various depths. This device consisted of a long, square rod which was marked off in 1-ft. lengths. At the lower end of the rod, and at right angles to it, was fastened a 1 x 3-in. strip slightly longer than the smallest inside dimension of the box. The end of the rod was at the center of the strip. This was duplicated exactly by a second piece, arranged so as to slide on the rod, which second piece always remained at the top of the box. By this arrangement, the top piece would always show the exact angular position of the bottom one, in whatever position the rod was twisted. By lowering the rod by degrees into the box in a fixed vertical line, and twisting it at each depth so that the bottom piece was "brought up" with its two ends against the two inner sides of the box, the slide at the top would give the exact angle of the bottom piece. Moreover, if one corresponding

DIAGRAM OF ARRANGEMENTS IN RETAINING-WALL EXPERIMENTS.

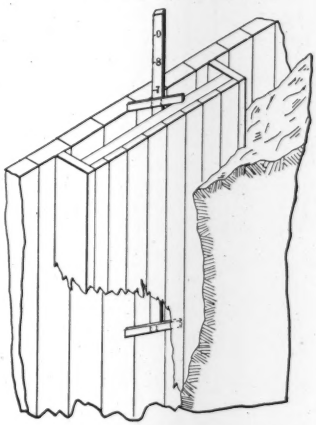


Fig. 10

DEFLECTION OBSERVATIONS.

TAKEN FEBRUARY 6TH

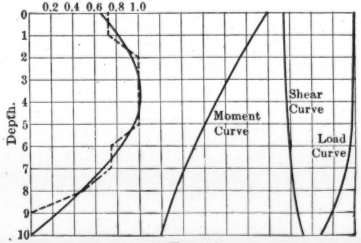


FIG. 11

TAKEN FEBRUARY 9TH

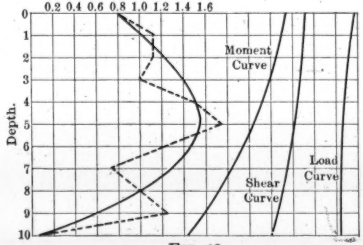


FIG. 12

TAKEN FEBRUARY 17TH

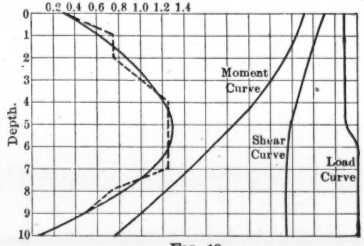


FIG. 13

TAKEN FEBRUARY 25TH

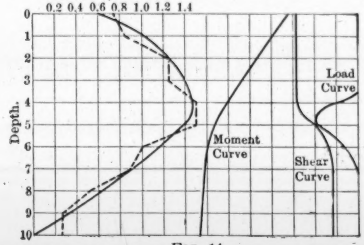


FIG. 14

TAKEN MARCH 10TH

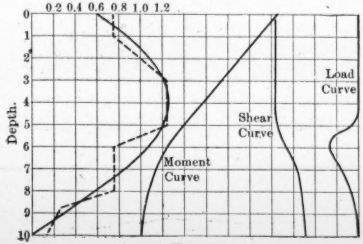


FIG. 15

TAKEN JUNE 9TH

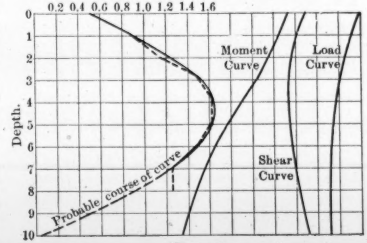


FIG. 16

end of the slide was brought in contact with one side of the box, at the top, the distance from the other end of the slide to the other side of the box would equal the difference in the breadth of the box between that at the top and that at the point of observation. This difference was the measurement sought, being the deflection caused by the earth pressure at the point observed. Readings were taken in this manner, (*a*) before any earth was in contact with the box, (*b*) as soon as it had been brought half way up the box, (*c*) as soon as the fill was completed, and (*d*) after varying intervals of time. The deflection curves are shown at the left of Figs. 11 to 16.

The filling consisted of earth excavated during freezing weather, and the fill was made by dumping on the edge of an embankment. Thus, the material immediately against the box at the time of the first two observations was principally frozen lumps. The third observation was taken after a thaw of several days, and the remaining ones after the frost had entirely left the ground.

It was desirable to obtain some analysis of the deflection curves, in order to see whether they would show in any way how the pressure obtained its full effect in deflecting the timber, and whether different conditions of the filling material caused any variation in pressure. The actual observations, therefore, were plotted for each series (shown by the dashed lines at the left of Figs. 11 to 16), and approximate deflection curves (shown in full lines) drawn through them. A curve in which each ordinate of one curve is proportional to the difference between two corresponding consecutive ordinates of another is the curve which is the first differential of the latter curve. In Fig. 17, let $a\ b\ c$ be the given curve, in which y_1 and y_2 are any two consecutive ordinates, all of which are spaced uniformly.

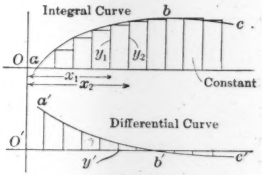


FIG. 17

By construction, make $y^1 = m\ (y_2 - y_1)$.

Then $$y^1 = n\,\frac{(y_2 - y_1)}{\text{constant}} = n\,\frac{(y_2 - y_1)}{(x_2 - x_1)} = (\text{at limit})\,\frac{d\,y}{d\,x}.$$

Hence, having drawn the deflection curve, the moment curve can be drawn by laying off equally spaced ordinates along the deflection curve and constructing a second curve in which similarly spaced ordinates are proportional to the difference between the consecutive

ordinates of the first curve. Similarly, a shear curve can be drawn from the moment curve, since the shear is the first differential of the moment at any point. The same rule applies to constructing the load curve from the shear curve, but, for a somewhat different reason. In Fig. 18, let $a\ b\ c$ be the shear curve, in which y_1, y_2 are any two consecutive ordinates, all of which are spaced equally. By construction, make $y^1 = m\ (y_1 - y_2)$.

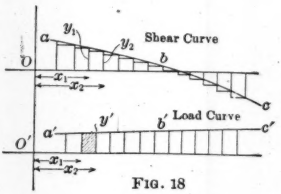


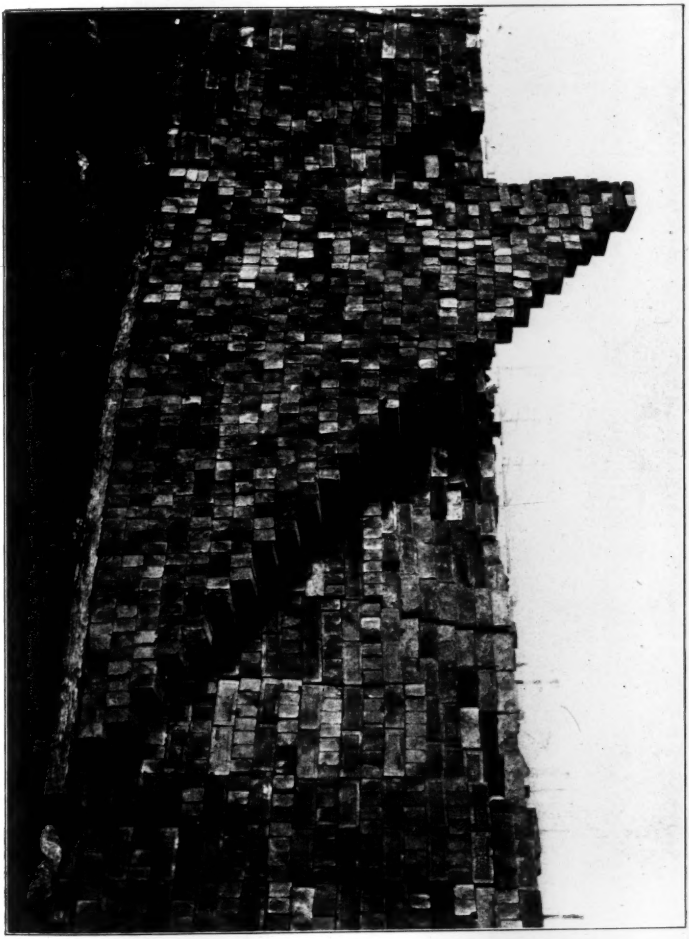
FIG. 18

Then $y^1 = n\ (y_1 - y_2) \div$ (constant) $= n\ (y_1 - y_2) \div$ (space passed over) $= n\ (y_1 - y_2) \div (x_1 - x_2)$.

Therefore, $y^1\ (x_1 - x_2)\frac{1}{n} = y_1 - y_2$. But, $y^1\ (x_1 - x_2) =$ area between the curve and the axis, which represents the load passed over. Consequently, the first curve is a shear curve for the second curve as a load curve; and conversely.

Moment, shear, and load-line curves were constructed in this way from the approximate deflection curves observed, and the load curves thus obtained afforded in some crude manner the information sought.

That the load is over only the lower half of the box, is shown by Fig. 11, and its practically uniform variation is revealed. Fig. 12 shows that immediately after the fill was completed the pressure extended throughout the full height of the box, and tends to show its approximately uniform variation. Furthermore, it has the slight curve near the surface found by Steel (see Fig. 5), and by the writer in his first-described experiment (see Fig. 9). The thaw which occurred between the second and third observations appears to have loosened the surface materials and to have brought into play an increased pressure at the upper levels (see Fig. 13). A week later the point of greatest pressure seems to have worked downward (Fig. 14), and, after another interval, appears to have dropped lower still (Fig. 15). In each of these three observations the local pressure was sufficient to relieve the plank from all strains, except in close proximity to the maximum point. After a final interval of three months, the load curve seems to have resumed its normal shape (Fig. 16), *i. e.*, practically uniformly varying. (Compare Figs. 12 and 16.) Of course, these deductions could be only qualitative in nature, but they do not appear abnormal, and are interesting in that they disclose in some



METHOD OF DETERMINING THE PRESSURE AND DEFLECTION IN SHEET-PILING.

measure the phenomena which take place during the gradual settlement into place of a mass of earth against a retaining wall, which deflects as the load increases.

In order to ascertain to some extent the amount of the pressure exerted by the earth in the observed cases, a 3 x 10-in. plank was supported at points 10 ft. apart and loaded with brick arranged so as to form a triangular pile with the apex over one support. The loading was continued, as shown in Plate XVI, until the observed deflection equalled that found in the earth-pressure observations. The weight of the brick thus piled was 2 580 lbs., which would be equivalent to a little more than 3 000 lbs. per foot of width of the earth. On the assumption of uniformly varying distribution, this gives a pressure of 600 lbs. per square foot at a depth of 10 ft. For various reasons, this seems to be excessive, and it is improbable that the actual pressure was more than 500 lbs. The material which made up the fill was principally fine beach sand which weighed about 100 lbs. per cubic foot in its final compact condition. This would show a lateral pressure approximately one-half of the vertical at the depths tested. This is large when compared with the observations thus far described, but, perhaps, may be accounted for partly by the action of the tide water, which was separated from the earth only by the 6-in. sheet-piling which formed one side of the experimental box.

In a similar manner, an attempt was made to ascertain the lateral pressure exerted by rip-rap under water. In this case, the pile of stone was 7 ft. deep and came only to a point 7 ft. from the top of the box. A deflection of $\frac{1}{8}$ in. was observed, and the weight of the brick necessary to produce an equal deflection under a uniformly varying load covering an equivalent portion of a 3 x 10-in. plank, 14 ft. long, was 452 lbs. This would mean a lateral pressure of about 150 lbs. at a depth of 7 ft. As the rip-rap (which was practically cobble stone) weighed about 140 lbs. per cubic foot in air, this would indicate a lateral pressure of slightly more than one-seventh of the weight in air, or, taking sea water at 64 lbs. per cubic foot, of nearly 30% under water. It is to be noted, on the other hand, as a fact within the writer's experience, that triangular piles of rip-rap about 14 ft. high readily withstand the thrust of a mass of earth 24 ft. in height above the same base (Fig. 19). On the assumption that the earth weighed 100 lbs. per cubic foot and gave a lateral thrust of one-

FIG. 19

half the load at any point, the total lateral pressure would be 28 800 lbs. On the further assumption that all this thrust was equilibrated by the rip-rap in such a manner that its lateral thrust varied uniformly (which is a forced assumption), the maximum value of this thrust would be 514 lbs. per square foot, found at a depth of 14 ft. Much rip-rap does not weigh more than 1 ton per cubic yard, or about 83 lbs. per cubic foot. Under water this would have an effective weight of only about 20 lbs. A pile 14 ft. high would exert a vertical pressure of 280 lbs. at its base. This amount readily withstands a lateral thrust of 514 lbs. Thus, while the least ratio, according to Moseley's principle, was 1 to 7 for the rip-rap tested, the greatest ratio may be almost 2 to 1.

Experiments with Testing Machine.

Description of Apparatus.—This actual value of the ratio of the lateral pressure to the vertical pressure is the important item in all questions involving earth pressure, and almost all queries concerning this subject can be answered satisfactorily when this ratio is known.

Several devices were constructed by the writer to measure this ratio directly, but, with one exception, without success.

It is an easy matter to provide and measure a vertical pressure upon a mass of earth, but, in order to measure the corresponding lateral pressure, some measuring instrument must be devised which will register the pressure and at the same time allow of no deformation under this pressure, for, the instant such a movement occurs, the internal conditions are altered and the results are rendered untrustworthy. An arching of the material will take place across any relatively small opening formed in the side walls of a containing vessel, so that the results usually obtained, when experiments are made with such an apparatus, do not show the true lateral resistance offered before the opening was made. It is well known that the pressures found against the valves of coal pockets and grain elevators are but a small fraction of those found against the entire sides of the pockets and bins.

An apparatus, Fig. 20, was finally devised which, it was believed, would provide against these difficulties. A cast-iron cylinder was bored out to a diameter of 6 ins. and a depth of 5 ins. It had a strong base and walls, and a plunger was fitted to it, which was arranged so that it could be secured to the head of an ordinary testing machine.

A 1-in. hole was bored in one side of the cylinder near the bottom, and a plug was carefully fitted to the hole. The outside of the cylinder was faced where the hole was bored, and the plug was made with a circular head, the underside of which was carefully turned to set accurately upon the planed portion of the outside of the cylinder. The plug was just long enough to reach through the wall of the cylinder and be flush with its inner face. The faced portion of the outside of the cylinder was slotted horizontally opposite the center of the 1-in. hole, and hard-rubber insulation was carefully fitted in the slots. Upon these hard-rubber pieces, thin strips of copper were secured and constructed so that they could be connected to the terminals of an electric battery. In the circuit with the battery was also placed an

CYLINDER FOR DETERMINATION OF RATIO OF LATERAL AND VERTICAL PRESSURES.

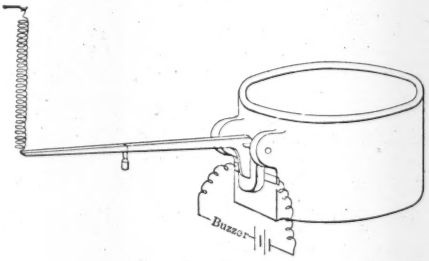


FIG. 20

ordinary buzzer. When the plug was fitted to the hole and pushed home, the underside of its head touched the copper strips on each side and thereby completed an electric circuit, the sounding of the buzzer giving evidence of the fact. If the plug was pushed slightly outward the circuit was broken and the buzzer stopped. Experiment showed that a movement of but $\frac{1}{1000}$ in. was sufficient to break the circuit, and that the tone given out by the buzzer was so sensitive that movements approximating $\frac{1}{5000}$ in. could be detected easily. With movements of the lateral weighing mechanism no greater than these, it is believed that no internal changes can take place which affect internal stresses sufficiently to vitiate the results obtained.

The cylinder was cast with two lugs upon the outside, just above the plug hole, and in such a position that holes bored through the lugs held a bolt which served as a fulcrum for a bent lever. One of the arms of the lever was horizontal and was marked somewhat like a

steelyard. The other arm bore against the head of the plug. Weights of various sizes were arranged so as to be hung on the horizontal arm and be moved along it as might be desired. The weight of the lever was neutralized by means of a spring attached to the free end of the horizontal arm and adjusted so that it relieved the head of the plug from all initial pressure.

Method of Experimentation.—In conducting an experiment, the cylinder was filled with the material to be tested, to a point slightly above the inner end of the plug, and placed upon the bed of a testing machine. The plunger was then adjusted under the head of the machine and brought down gradually upon the material in the cylinder. About the time that contact between the disc and the material was expected, the buzzer was put in operation and gave forth a continuous sound, until the lateral pressure against the inner end of the plug was sufficient to push it out slightly and break the circuit. The increasing application of the vertical pressure was then stopped and the location and value of the weights upon the horizontal lever arm were noted. The lateral pressure against the head of the plug was then carefully increased, by changes in the location and amounts of weights on the horizontal arm, until this external lateral pressure was just sufficient to press the plug back to its original position and thus restore the electric circuit and start the buzzer. The testing machine was then again started, and the vertical pressure increased until the internal lateral pressure was sufficient to move the plug outward and stop the buzzer, thus completing a cycle of operations. This series was repeated until any desired limit of vertical pressure had been reached. Usually, after such an increasing series had been executed, the reverse operation was followed through, as the machine was being brought back to a zero reading. This showed the difference in the action of the material, in regard to its lateral thrust, during increase and decrease of vertical load.

It was only after considerable experiment that the above-described method of procedure was decided upon. The lower curve of Fig. 21 shows a typical series of results obtained by it. For comparison, there are also shown in the upper curve the results secured when the load on the testing machine was reduced each time, so as to start the buzzer again after each new increment of pressure had been brought to bear on the head of the plug, and the vertical pressure had been worked up to the point of stopping the buzzer. Asterisks show

where a new weight was placed on the horizontal lever arm, and are the points at which breaks in the curve would be expected to occur, if they are to occur at all. In all that follows, the curves given and described are those formed by using the first of the above-described methods, and only the increasing series of readings are treated, unless expressly stated to the contrary.

Discussion of Results.—While many questions arise, concerning the reliability of the results obtained and their proper interpretation, several points seem to be brought out fairly well by the observations.

The effect of repeated applications of pressure without loosening up the earth between such repetitions is shown in Fig. 22. It is to be observed that in dry materials the curves usually remain practically straight lines, differing only in slope and always starting from zero. In moist earths the first application of pressure, also, usually gave a straight line, but a peculiar permanent set sometimes seemed to be encountered in succeeding applications of pressure during their earlier portions. This is revealed clearly by the curve which forms the upper part of the line of pressure, in the 30–50 sand observation. It starts from zero, but attains the direction of its later movement only by a reversed curve. This starting curve shows that, in some instances, after the first application of vertical pressure, lateral pressures may exist which are larger than the corresponding vertical ones. (See Fig. 23, which shows the second application of pressure on 30–50 sand at 31% of saturation.) This only serves to confirm the observations made by Darwin with his model, that previous treatment, and, in his case, the direction of even slight stratification, affects materially the amount of the lateral pressure. This peculiar action of moist earth is of interest to designers of quay walls, dock walls, dock bulkheads, dry-dock walls, etc., who have to deal with cases where excessive loads must be borne occasionally. In general, however, when the earth is not touched between successive applications of pressure, the change of slope of the lines is such that the actual lateral pressure, up to a certain limit, is slightly less with each successive application. This is in conformity with the observed fact that freshly made fills behind retaining walls seem to exert greater lateral pressure than they exert after an interval of time. Of course, this effect of diminished pressure may also be caused by a gradual drying out of the fill, and sometimes by other causes, but it is

probable that repeated applications of heavy loads, also, have some effect.

The tests of dry materials, which differed only in the size of the particles, seem to reveal a singular increase and then a decrease of the ratio of lateral to vertical pressure. The maximum ratio is found with 30–50 sand; that is, with sand consisting of particles which pass through a sieve having 30 meshes per linear inch, or 900 per square inch, and are held on one having 50 meshes per linear inch, or 2 500 per square inch. (See Fig. 24.) The erratic nature of the curves for ½-in. and ¼-in. gravel (Figs. 25 and 26), show primarily that the size of the individual grains was too large for the special mechanism used to test them. If, however, the first three observations for ½-in. gravel and the general trend of the results for ¼-in. gravel are used for comparative purposes, a most interesting compound curve is developed. (See Fig. 27, where observations on rip-rap, made by the writer, are also included for wider comparison.) With very coarse particles, such as rip-rap and coal, the ratio is small, but increases as the materials grow smaller, through ½-in. and ¼-in. gravel, to the coarsest sand. When the particles range in size from those which will just pass a sieve with 20 meshes to the inch to those which will just be held by a sieve with 50 meshes to the inch, the ratio reaches a maximum. As the particles diminish in size still further, whether they are uniform or vary so that the mass contains large and small grains, with an appreciable percentage of the latter, the ratio diminishes. In Fig. 27 are plotted the ratios observed at 6 000 lbs. per square foot vertical pressure, for all the sizes of grains tested. The curve thus formed seems to start from zero and increase rapidly to a maximum, then diminish and, finally, with a change in curvature, go out to infinity along one axis as an asymptote. This makes a curve astonishingly like the lituus shown in the same figure, but no relation is apparent between the equation of the lituus and the conditions involved in this problem. A little consideration, however, shows that this curve does represent about what is to be expected. In masses formed of very large pieces there is usually little lateral pressure, and when the number of particles per inch approaches zero, that is, when the pieces combine in a single block of stone, the lateral pressure, of course, is zero. It is evident, at the other extreme, that, as the size of the granules grows smaller until they become microscopic, constantly increasing cohesive molecular forces will come into play, so that, with particles of molecular dimen-

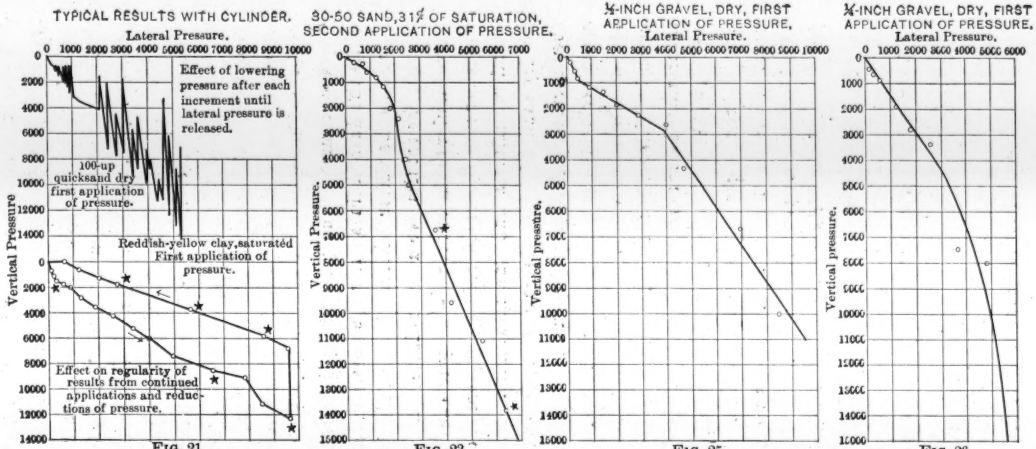


FIG. 21 FIG. 23 FIG. 25 FIG. 26

CURVES SHOWING RELATION OF RATIOS UNDER REPEATED APPLICATIONS OF PRESSURE.

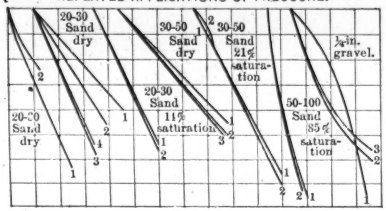


FIG. 22

CURVES SHOWING RELATIVE RATIOS FOR DRY MATERIAL

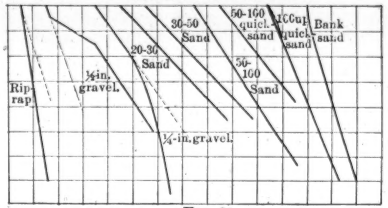


FIG. 24

sions, the mass has become again one solid block through cohesion, and can exert no lateral pressure of the variety studied herein. Baker's results also confirm the conclusion stated above, to some extent.

Concerning the effect of moisture upon the ratio of lateral to vertical pressure, an even more surprising phenomenon is observed. See Figs. 28, 29, and 30. For each class of material, four curves or parts of curves are there shown, which represent the lateral pressures observed at different percentages of saturation, under vertical pressures of 2 500, 5 000, 7 500 and 10 000 lbs. per square foot. Beginning with the lowest one and counting upward, the curves are for these pressures, respectively. In these diagrams the actual observations made are shown by the small circles. The full-line curves show the probable course of the phenomena between the actual observations, while the dashed lines show the possible remaining portions of these curves, as the writer believes they would be. While not absolutely proven, the results seem to indicate that, in all the cases tested, a curve similar to the one for 50–100 quicksand is to be expected, and that the latter may be considered as a typical one. In this curve are to be found two minima and one maximum. Starting with a given ratio for dry sand, the ratio decreases at first with increasing saturation, then increases sometimes beyond the starting value, only to decrease again nearly as low as in the first minimum, and finally to increase up to a final medium value. This is not quite in accord with the curve given by Wilson, but his observations covered only two percentages of saturation, aside from the dry and practically saturated, so that it is still possible that a curve of the form found by the writer may be drawn through the values found by him. This is shown in the diagram at the right of Fig. 30. Steel's results are not of a nature to afford deductions as to this matter. On the other hand, the last deduction made by Wilson is borne out by the writer's work, as the ratio of lateral to vertical pressure was found by both observers to increase with increase of pressure. (Compare Table 1 with Fig. 6.)

These curves for varying percentages of moisture may be represented by equations of the form,

$$y = a + b\,x + c\,x^2 + d\,x^3 + e\,x^4,$$

in which the coefficients may be determined for any given set of observations, but such determinations are tedious and the resulting quantities indicate nothing as to the questions naturally arising. The

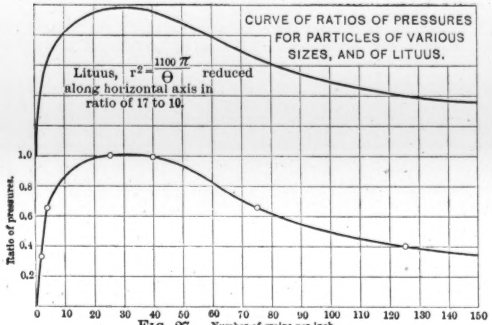


FIG. 27

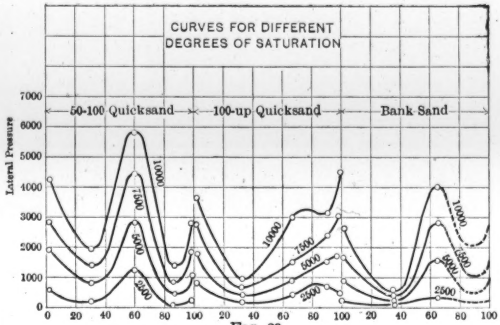


FIG. 29

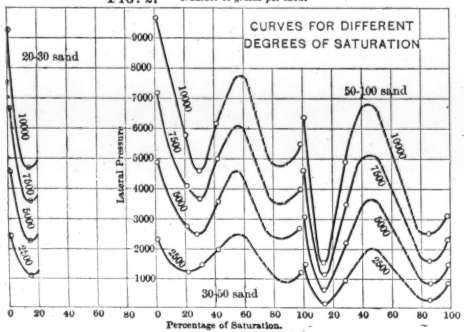


FIG. 28

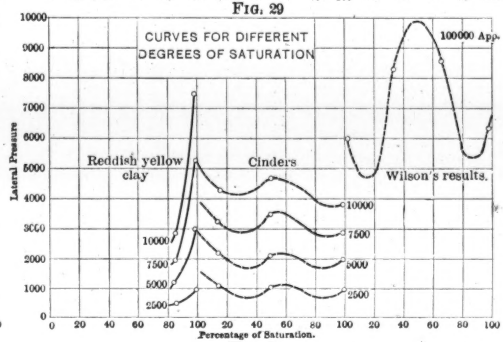


FIG. 30

equation which will give practically the curve for 50–100 quicksand under a pressure of 10 000 lbs. per square foot (see the upper left-hand curve in Fig. 29), is

$$y = 0.00001056\,x^4 - 0.016754\,x^3 + 1.4056\,x^2 + 16.28\,x + 42,$$

in terms of hundreds of pounds vertical pressure, measured along the axis of y, and percentages of saturation, along the axis of x.

Why the equation should be of this form is not obvious, but a possible explanation of the shape of this typical curve may be made. It is well known that with slight additions of moisture the cohesion between the particles of earth is increased greatly, and hence the ratio of vertical to lateral pressure would probably decrease down to some limit. (See Fig. 31.)

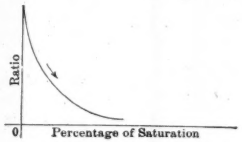


FIG. 31

It is well known, however, that, long before the moisture would fill all the voids, some change takes place to reduce the cohesion and hence increase the ratio of vertical to lateral pressure. (See Fig. 32.) At the other extreme, it is well known that considerable water makes an excellent lubricant between polished surfaces of stone, and that the friction increases with decrease of water. As the friction would increase, the cohesion would increase and the ratio of the vertical to the lateral pressure would decrease. (See Fig. 33.) Experience, however, teaches that when the amount of moisture is very small, the friction is also small and, similarly, the ratio of vertical to lateral pressure would be relatively larger. (See Fig. 34.) The curves produced by these two causes might coincide, for aught one can see,

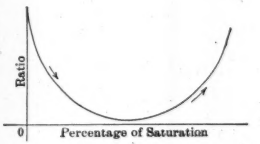


FIG. 32

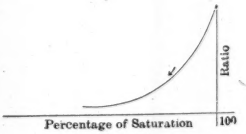


FIG. 33.

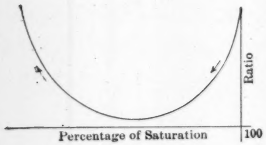


FIG. 34

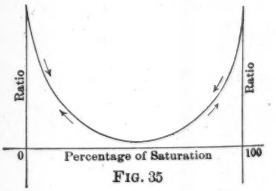


FIG. 35

but the experiments seem to show that they do not, but form the double curve actually observed. (See Figs. 35 and 36.) Inspection of the various saturation curves, further, shows that the effect of the moisture is more marked under heavy vertical pressures than under lesser ones.

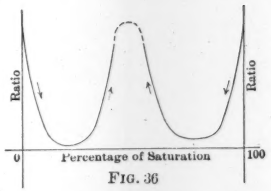


FIG. 36

The large ratios found for 20–30 and 30–50 sand are suprising, and would be open to more doubt, but for the singular regularity of the curve of Fig. 27. Perhaps, however, this is only an accident. Comparison of the curves for 50–100 bank sand and 50–100 quicksand reveals great similarity, and suggests that it is the fine particles only of quicksand which give it its peculiar qualities. The curves for 100–up quicksand do not show a well-marked second minimum, but do show a fairly constant increase of lateral pressure, up to saturation. At this saturation point, the lateral pressure is larger than for materials of any other size, except the surprising 30–50 sand and the clay. Experience would lead one to expect a relatively high lateral pressure for the latter material, especially when saturated. But, in the quicksand, even with the relatively high lateral pressures at saturation, they do not approach nearer than about 50% of the vertical ones when the water of saturation is confined with the sand, as occurred in the model during the test, and these lateral pressures are much less when the water is drained away to some extent, that is, when the sand is not saturated. Therefore, it would seem that those designers who are using a ratio of one (a "hydrostatic" ratio) in designing retaining walls where quicksand is encountered are using excessive assumptions. Apparently, at least in part, the peculiar nature of quicksand inheres in its extreme fineness. More than 80% of the sample from the Brooklyn Navy Yard tested by the writer passed a standard No. 100 sieve. This property makes it practically impossible to confine it. It will flow, when saturated, wherever water does. Furthermore, it is doubtful if the large pressures sometimes attributed to quicksand are really more than those actually produced by the water saturating the sand. The problem of the designer, dealing with quicksand, is only, then, to impound completely the water which the sand contains, and in accomplishing this he need deal with no pressures except the hydrostatic ones.

TABLE 1.—AVERAGE RATIOS FOR VARIOUS MATERIALS AND PRESSURES.

Kind of material.	Observer.	Average Ratio of Vertical to Lateral Pressure.				Variation from Average Ratio of—								Remarks.
						Maximum probable value.				Minimum probable value.				
		2 500 lbs.	5 000 lbs.	7 500 lbs.	10 000 lbs.	2 500 lbs.	5 000 lbs.	7 500 lbs.	10 000 lbs.	2 500 lbs.	5 000 lbs.	7 500 lbs.	10 000 lbs.	
		Ratio.	Ratio.	Ratio.	Ratio.	Per-centage.	Per-centage.	Per-centage.	Per-centage.	Per-centage.	Per-centage.	Per-centage.	Per-centage.	
Coal, shingle, ballast, macadam materials	Baker	0.10	.....	.....	.....	6	.....	.....	.....	4	.....	.....	.....	
Rip-rap (under water)	Goodrich	0.15	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	
½-in. gravel	"	0.41	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	Observations incomplete.
¼-in. gravel	"	.....	.....	.....	0.50	.....	.....	.....	.....	.....	.....	.....	.....	Observations incomplete.
Hard-coal cinders	"	0.40	0.42	0.44	0.44	4	2	5	2	0	4	4	6	
20–30 sand	"	0.72	0.71	0.72	0.71	.....	.....	.....	.....	28	25	24	24	Observations incomplete.
30–50 sand	"	0.60	0.60	0.60	0.60	.....	.....	.....	.....	12	10	11	14	Observations incomplete.
50–100 sand	"	0.36	0.40	0.40	0.40	.....	.....	.....	.....	32	28	25	24	Observations incomplete.
50–100 quicksand	"	0.24	0.26	0.30	0.32	34	32	26	16	10	14	22	28	
100–up "	"	0.20	0.22	0.25	0.28	12	14	14	18	12	10	16	17	
Bank sand	"	0.10	0.18	0.24	0.25	2	14	13	15	2	12	18	19	
Earth	Steel	0.26	.....	.....	.....	10	.....	.....	.....	10	.....	.....	.....	
"	Baker	0.15	.....	.....	.....	5	.....	.....	.....	5	.....	.....	.....	
Bank sand	Wilson	.....	.....	.....	0.26	.....	.....	.....	10	.....	.....	.....	10	
" "	Goodrich	0.33	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	With friction apparatus.
" "	"	0.50	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	Against retaining wall.
Clay	"	0.40	0.40	0.40	0.40	20	20	30	36	.....	.....	.....	.....	Observations incomplete.
"	Baker	0.22	.....	.....	.....	35	.....	.....	.....	10	.....	.....	.....	

The high lateral pressures found in absolutely dry sand made up of coarse grains are of no importance to designers, as such materials are never encountered in practical experience, and no materials are found in a loose, dry state. Whenever earth dries out completely, the particles are cemented together into a fairly rigid mass by the evaporation of the moisture which the earth contained and the deposition of the solids which were held in solution in it. It then exerts little lateral pressure, or none at all.

With the usual factors of safety for engineering structures, it is not necessary to allow for the maximum lateral pressures observed for any kind of material, but a selected average value can be used without dangerous error. Such an average, when selected carefully, will give results within the range of variation due to other causes. Table 1 1 shows such values, collated from all the foregoing data, and also shows the percentage which the maximum and the minimum probable values will differ from this average.

The observations of the writer upon his 3-ft. model with bank sand are omitted purposely, as the results, evidently, were influenced by the small size of the model. It is doubtful whether models less than 6 or 10 ft. in least dimension will ever afford trustworthy results, except with perfectly dry materials.

The ratio found for rip-rap by the writer agrees fairly well with the ratio deduced from Baker's paper for materials of somewhat similar character. The values for gravel and cinders also have fair uniformity. The unexplained decrease of the ratio with increased fineness of material is again noticeable in Table 1, as well as the facts that, with coarse materials, the ratio is practically constant, whatever the pressure, and that when the material is fine in texture it increases with the pressure. Perhaps the latter fact can be explained partly by the increased compressibility of the finer materials, and, consequently, the increased internal friction which would be expected when the particles are close together.

The low ratios of 0.10 and 0.07, for bank sand, found by the writer, with the cylinder and the model, do not accord well with those of 0.26, 0.33 and 0.50 found by Steel for earth and Wilson for sand, and by the writer for bank sand in the retaining-wall experiments. The values of 0.07, 0.10, 0.15, 0.26, 0.33 and 0.50, made up of those mentioned, together with one of Baker's for earth, show conclusively, however, that a general average cannot be determined which will not be subject to a

possible 50% error, and that for a close design one must examine carefully the kind of material, and ascertain, as well, its degree of saturation and its previous treatment. After the determination of these points, reference would best be made to the curves of Figs. 28, 29, and 30. Baker's maximum ratio for clay is practically the same as the writer's minimum, and, because of the character of the observations, the agreement can be said to be good.

For almost all purposes, the ratios in the column headed 2 500 lbs. will be the ones to be used, because, with earths of common density, such a pressure would be found at depths of about 25 ft.

From the data in Table 1, it is evident that a ratio of 40 : 100 is ample for all but exceptional cases, and that, where conditions can be regulated, a ratio of still smaller size can be used. The actual relation which the use of this ratio bears to common practice is shown clearly by Fig. 37.

Let a wall h feet high and b feet thick, weighing 150 lbs. per cubic foot, be made to support a uniformly varying earth pressure wherein the lateral thrust is four-tenths of the vertical. Let this be due simply to the weight of the earth, which is taken at 100 lbs. per cubic foot. The usual rule is followed, of applying the resultant lateral thrust at one-third the height of the wall from its base, and of allowing the resultant of the pressures to cut the base at the edge of its "middle third."

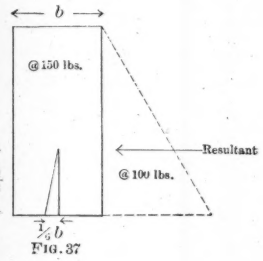


FIG. 37

Then, lateral pressure $= \frac{4}{10} \times \frac{100\,h}{2} \times h = 20\,h^2$;

Total weight of wall $= 150\,b\,h$.

Then, $$20\,h^2 : 150\,b\,h :: \frac{1}{6}\,b : \frac{1}{3}\,h,$$

$$\frac{20}{3}\,h^3 = \frac{150}{6}\,b^2 h,$$

$$4\,h^2 = 15\,b^2,$$

$$b = h\sqrt{\frac{4}{15}} = 0.484\,h = \frac{h}{2}, \text{ approximately.}$$

Trautwine* gives 0.5 as the relative thickness for walls of "well-scabbled dry rubble," with ratios of 0.4 and 0.35 of the height for

* Engineer's Pocket-Book, 18th ed., p. 603.

breadth of walls of "good common scabbled mortar-rubble, or brick" and of "cut-stone, or of first-class large ranged rubble, in mortar," respectively. These ratios tend to show that common practice either does not limit the resultant to the "middle third," or assumes a ratio of lateral to vertical pressure less than 0.4.

Point of Application of Resultant.

The lateral pressures obtained by Steel, and several of the writer's results, show a departure from uniform variation of which a few authorities try to take account. Where the curves are straight lines starting from zero, the resultant would be applied, theoretically, at a point one-third of the height of the plane of pressure above its bottom. Where the curves are concave upward, as in the second and third applications of load to the ¼-in. gravel, and the first observations on 20–30 sand (see Fig. 22), the resultant would be found below the one-third point. This condition is rarely met, and may be neglected. In many cases, however, the curve is convex upward, and the resultant would actually be located above the one-third point. This is on the danger side, if the common rule is followed, of basing computations on the assumption of its being applied at the one-third point. While the reliability of the curves here shown is rather small, in many cases, still an investigation of them along this line will prove of interest. In the following-named curves, the location of the resultant is as indicated:

Figure.	Fraction of height above bottom at which resultant is applied.
3	0.40
4	0.39
5	0.38
9	0.38
12	0.40
16	0.39

This would tend to show that a much safer practice, in walls from 6 to 10 ft. high, would be to assume that the resultant is applied at a point four-tenths of the height from the bottom, in place of one-third, as is usually done at present. The difference between these two assumptions involves the stability of structures to the extent of 20 per cent. With walls less than 6 ft. high, the resultant should be applied still higher; while, with walls more than 10 ft. high, the point of application will approach the one-third point more and more as the height of wall increases.

Internal Friction Angle.

According to Rankine*, the ratio of the vertical and lateral pressures is expressed by the ratio,

$$\frac{1 - \text{sin. } \phi}{1 + \text{sin. } \phi},$$

in which ϕ is the limiting angle of internal friction at any point. If the ratio is known, the angle can be computed from the formula,

$$\text{sin. } \phi = \frac{1 - r}{1 + r}.$$

Having found the sine of the angle, its tangent can be found easily in any table of natural functions.

Table 1 shows that the ratio varies with the pressure, in many cases, and, consequently, the limiting angle of friction must vary accordingly. Similarly, this limiting angle of internal friction is certain to differ in most cases from that of the angle of the natural slope of the free surface of the material.

This angle of slope was determined carefully for bank sand of varying percentages of saturation, by dropping it carefully from a given height (of a few inches) so that it would form a cone, and then measuring the altitude and average diameter of the base of this cone, and computing the tangent of the angle of surface slope. The results thus obtained are shown in the curve at the left of Fig. 38. For purposes of comparison, the tangent of the limiting angles of internal friction, in the case of 50–100 quicksand, were computed from the formula, and in the manner described above, and are shown in the other curves of Fig. 38. It is somewhat surprising to find the same type of curve with a maximum and two minima in the case of the surface angle, but its nature is evident and also its general similarity to the curves of the computed tangents.

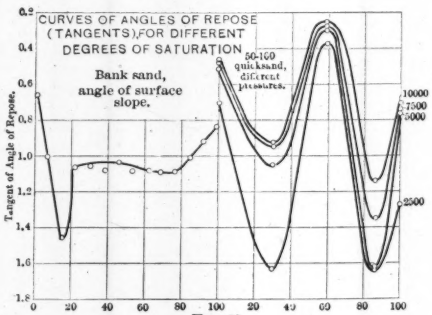


FIG. 38

* Civil Engineering, 19th ed., p. 319.

The actual surface slope of the bank sand tested, in connection with the retaining wall, was also found on the occasion of a slip which occurred. The slope under water was 1 to 2, and above water level 1 to 1. The surface slopes of the samples of cinders, gravel, etc., experimented with were also determined, and are given later.

A series of experiments was also carried out to determine the actual tangent of the limiting angle of internal friction for various earths. That it varies with the pressure, is shown readily by constructing two boxes without tops and of the same dimensions except the depths, which must differ. Fill both with sand and put one above the other with the open tops together, then tilt the under one until the upper one just slides, and then repeat the operation with the boxes reversed. A marked difference in the angle of sliding will be observed with damp sand, even with boxes but 2 and 3 ins. deep.

Quantitative experiments of this character were carried out by building two boxes, 1 ft. in each internal dimension, but without tops and only one with a bottom. (See Fig. 39.) The latter box was fastened to the edge of a high platform and had a pulley attached to it, and arranged so that a line would lead from the other box when placed on top of the first, over the pulley to a scale-pan upon which known weights could be placed. The two boxes were placed in position, and the weight which would just move the upper one over the fixed one when empty was ascertained. The coefficient of friction of the pulley was also determined for different loads, and the results were used to correct the later observations.

MODEL FOR DETERMINATION OF COEFFICIENT OF INTERNAL FRICTION.

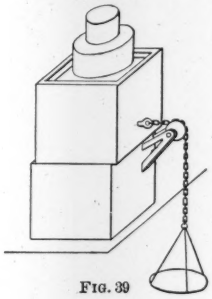


FIG. 39

In conducting an experiment, the two boxes were filled with the earth to be tested and enough weights added to the scale-pan to start the upper box to sliding. It was then checked and jacked back into position and a loose top placed on the upper surface of the earth and known weights piled on this top. Weights were then added to the scale-pan until sliding again just took place. This operation was repeated with increasing weights on the earth, until, for various practical reasons, it became impossible to carry the operation farther.

Fig. 40 gives the actual results obtained, and, for comparison with previous results, Fig. 41 shows the results of the observations reduced so as to give the lateral pressures for various loads. The writer's observations with the 3-ft. model are inserted, for further comparison. Figs. 42, 43, 44 show the relation between the lateral pressures computed from the friction experiments and those observed directly with the cylinder. They show conclusively that Rankine's theory of conjugate pressures is correct when the proper angle of friction is used.

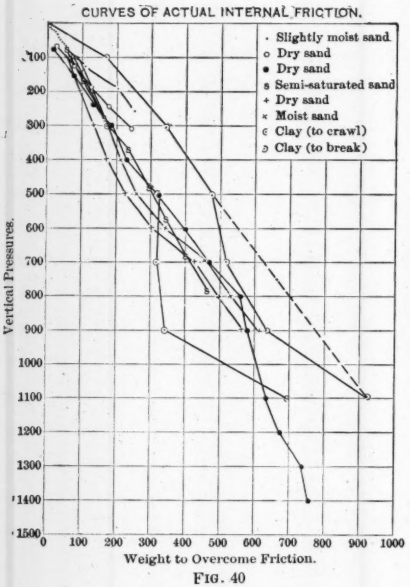


FIG. 40

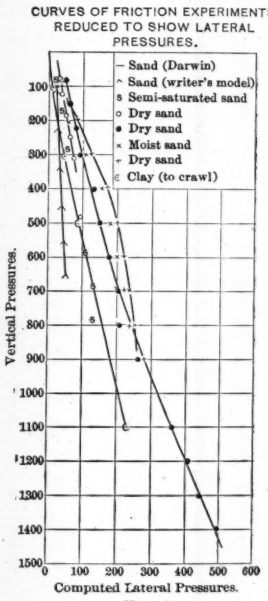


FIG. 41

The relation between the angle of surface slope and the angle of internal friction in one instance is shown in Fig. 45. The dashes show the possible connecting curves between those of the internal friction angles and the points locating the tangents of angles of surface slope. It would appear from this figure, and from the curves of Figs. 42, 43, and 44, that some relation exists between the angles of surface slope and of internal friction. This relation is shown in Fig. 46, where the dry-sand curve of Fig. 45, slightly altered, is reproduced.

(Curve *A*.) Both curves of Fig. 45 are also drawn to one-tenth the horizontal scale there shown, with further observations inserted. (Curves *B* and *C*.) The dry-sand curve is also again drawn to one one-hundredth

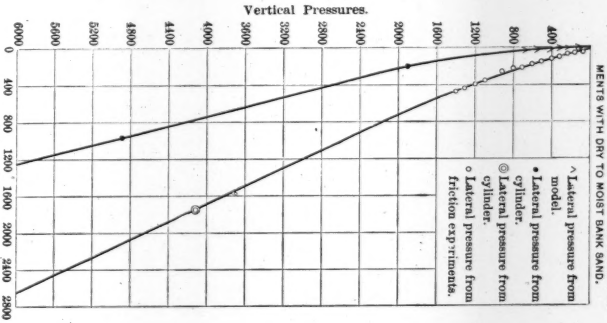


FIG. 42.

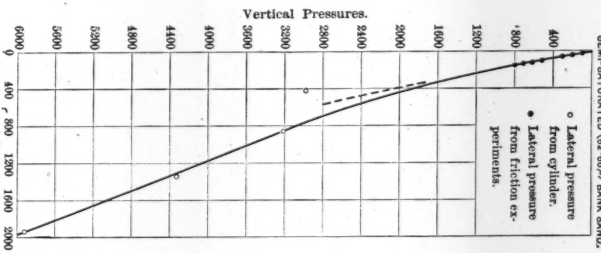


FIG. 43

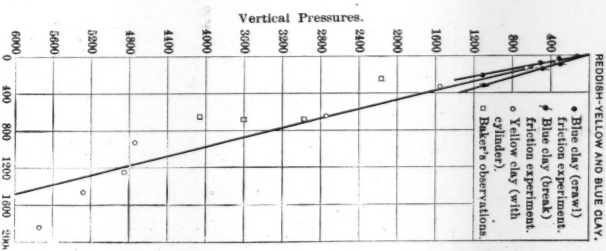


FIG. 44

of that scale horizontally, with still further observations inserted. (Curve *D*.) For all practical purposes, the curve may be considered as a nearly straight line composed of the right-hand main portion of the curve, and the dotted curve connecting the starting point with this main portion. The sharp drop to the minimum is of no consequence,

as it occurs in the first foot or two of depth. In this special case, for depths down to about 6 ft., the tangent of the angle of internal friction may be considered as that of the angle of surface slope. Below

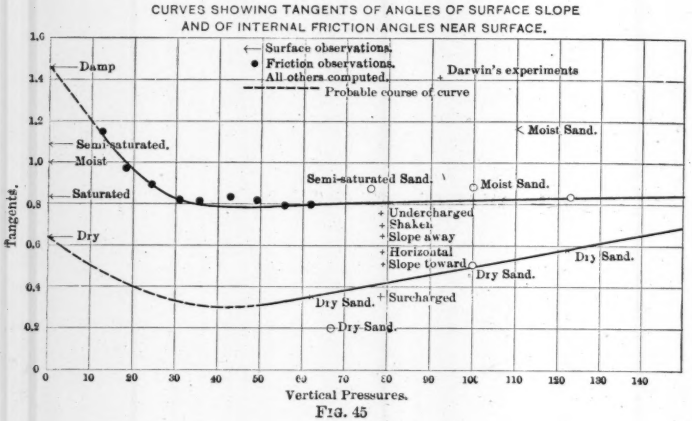


FIG. 45

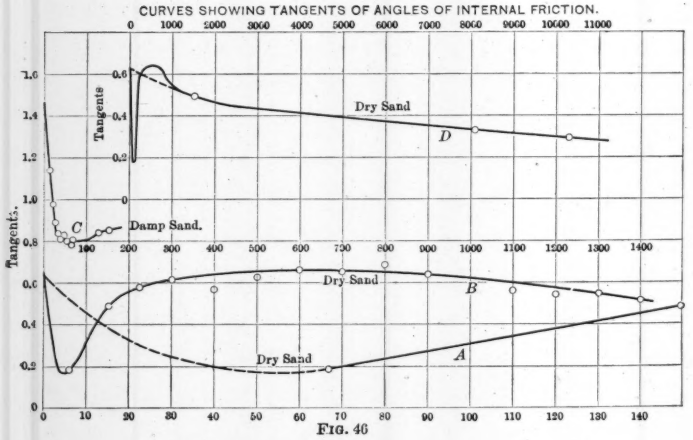


FIG. 46

6 ft., however, the angle of internal friction falls off, at first rapidly, then more and more slowly. The determination of an analytical curve of similar shape can be of no practical value, as will be shown directly.

TABLE 2—ANGLES OF INTERNAL FRICTION AND OF SURFACE SLOPE FOR DIFFERENT MATERIALS.

Ratio of lateral to vertical pressure.	Kind of material.	Observer.	Tangent of angle of internal friction.	Tangent of angle of surface slope.	Observer.
Percentage.					
10	Coal, shingle, ballast, etc.	Baker	1.423	1.11 to 0.70	Rankine.
				1.45 " 0.60	Goodrich.
	Bank sand	Goodrich	1.423	1.00 " 0.67	Trautwine.
				0.75 " 0.38	Rankine.
15	Rip-rap	Goodrich	1.097	1.00	Goodrich.
	Earth	Baker	1.097	0.66	Trautwine.
20	100-up quicksand	Goodrich	0.895		
	Clay	Baker	0.895	1.00	Rankine.
	50-100 quicksand	Goodrich	0.750		
25	Earth	Steel	0.750	0.66	Trautwine.
				0.58	Steel.
	Bank sand	Wilson	0.750	0.58 to 0.62	Wilson.
30			0.622		
35	50-100 sand	Goodrich	0.549	0.85	Goodrich.
	Bank sand	"	0.549	0.75 to 0.38	Rankine.
40	Clay	"	0.474	1.00	Rankine.
	Cinders	"	0.474	0.86	Goodrich.
	½-in. gravel	"	0.474	0.66	Trautwine.
50	¼-in. gravel	"	0.350	0.85	Goodrich.
	Bank sand	"	0.350	0.75 to 0.38	Rankine.
60	30-50 sand	"	0.258	0.66	Goodrich.
70	20-30 sand	"	0.179	0.75 to 0.38	Rankine.

Table 2 gives various ratios of lateral to vertical pressure, the materials with which the experiments were made, and the computed tangent of the limiting angle of internal friction. The tangents of angles of usual surface slope are also included, but the relation between these two angles is shown more clearly in Fig. 47.

DIAGRAM SHOWING RELATION OF TANGENTS OF ANGLES OF SURFACE SLOPE AND OF INTERNAL FRICTION.

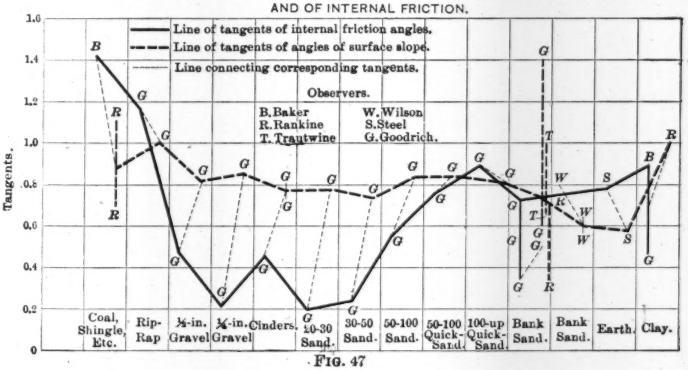


FIG. 47

With the exception of rip-rap and clay (the particles of the one being perhaps too large and of the other too small), all the surface slopes tend toward such as have tangents between 0.50 and 1.00, irrespective of the angles of internal friction. This would tend to prove that practically the same laws of rolling friction exist for materials of different-sized particles and degrees of saturation, while the laws of internal friction depend more intimately upon, and vary with, the size of the particles and the percentage of their saturation. Consequently, designers must look to the angle of internal friction and not to that of the surface slope to furnish them with data for their computations. These vary considerably, under different circumstances, and no general rule can be laid down.

Compressibility of Soils.

Tests of the compressibility of various soils were also made, but they revealed only the facts: (*a*) that the layers immediately beneath the compressing disc were much more compressed than the more distant ones; (*b*) that a sort of compressed cone formed under the compressing disc; (*c*) that an appreciable percentage of compression took place only with excessive loads which would correspond to great depths below the surface; (*d*) that, under extreme pressures, garden earth and sand showed quite an elastic reaction, the former showing the more, and (*e*) that clay, even when showing but a small quantity of water present, would "crawl" and relieve the pressure by squeezing through the smallest openings in long threads or sheets.

Recapitulation.

More experimental data of a practical variety should be secured, as the subject of pressures in earth is one upon which the whole scientific design of the foundations of engineering structures depends.

It is believed that the scientist can work along the lines described herein with a more or less clear perception of the following facts or conclusions, herein first deduced, or here brought out in confirmation of what experience has already taught:

1.—Repeated applications of pressure to earths in general develop diminishing lateral reactions.

2.—In moist earths the first large application of pressure is likely to produce a permanent set which exerts excessive lateral thrusts at low repetitions of pressure.

3.—The lateral thrust in dry materials which differ only in size of particle varies as the ordinates to a lituus.

4.—The lateral thrust, in materials which differ only in the percentage of saturation, varies as the ordinates to a curve of the fourth degree, possessing two minima and one maximum between 0 and 100% of saturation.

5.—Some explanation of the lituus curve may be obtained from the relation of molecular forces to the size of the granules.

6.—Some explanation of the curve of the fourth degree may be obtained from the phenomena of adhesion between water and particles of earth, together with capillary action, and of the friction between two faces of rock with water as a lubricator.

7.—The effect of moisture is more marked at heavy vertical pressures than at lesser ones.

8.—The peculiar nature of quicksand inheres, in part at least, in the size of its particles, and the curves of the fine portion do not possess the maximum to be found with particles of larger sizes.

9.—Clay, which possesses still smaller particles, has a higher relative lateral thrust, but, because of its nature, does not "flow," but "crawls."

10.—Earth-pressure experiments, to be of practical value, must be on models of large size, because of the effect of arching action.

11.—The effect of increase of pressure is more apparent in materials of small particles. Perhaps this is because of increased internal friction. The greater compressibility of fine materials results in a relatively large decrease in the distance between particles and thus in increased friction.

12.—In many cases the resultant of the pressures on a plane should be applied at a point above the lower one-third point.

13.—Rankine's theory of conjugate pressures is correct, when the proper angle of friction is found, and probably adaptations of his formulas will be of most practical value.

14.—The coefficient of rolling friction of particles of different sizes is practically constant, irrespective of size and of amount of moisture present.

15.—The limiting angle of internal friction depends upon the size of the particles and the degree of saturation of the material.

16.—In the loading of earth, a cone of compressed material forms under the compressing body.

17.—Earths, under certain conditions, reveal elasticity.

For the practical engineer and the designer of structures, the following points are deemed pertinent:

1.—The lateral thrust of newly made fills against retaining walls decreases with time and repeated applications of the load.

2.—With some moist earths, a heavy load will induce in the material a possibility of the development of excessive lateral thrusts under repetitions of even relatively small loads.

3.—Neither saturated nor loose dry materials are apt to exert the greatest lateral thrust, and, with slight saturation and rather moist conditions, the lateral thrust is apt to be relatively small.

4.—When dealing with quicksand, the problem is to confine the water which it contains, and account need be taken of pure hydrostatic pressures only.

5.—When dealing with clay, the problem is to remove the water which it contains and guard against the creeping of the material under excessive loads.

6.—In many cases the resultant of the lateral pressures on a plane should be applied at a point fourth-tenths of its height above the bottom.

7.—Angles of internal friction and not of surface slope must be used in all formulas which involve the sliding of earth over earth. Such angles, or the ratio from which they may be computed, are given in the writer's tables and curves.

Acknowledgment is due, especially, to W. L. Sturges, A. I. E. E., for interest and assistance in securing data for the materials herein contained, and to the Brooklyn Polytechnic Institute for the use of its testing machine.

DISCUSSION.

M. F. Bonzano, M. Am. Soc. C. E. (by letter).—There has been too little investigation of this important subject, and it is gratifying to have Mr. Goodrich's paper. The writer has for years looked forward to having an opportunity to make some investigation with a view of securing some reliable data upon this subject. Mr. Bonzano.

The writer has noticed that the shrinkage of dumped material is in the direction in which it flows when dumped, and that the principal lines of pressure are substantially in the same direction, and, as a result, has introduced into specifications for graduation the following:

"In depositing filling material against abutments, piers, or walls, it must always be dumped away from masonry; never toward it. As the shrinkage of material is always in the direction in which it flows when dumped, especial pains must always be taken to make the shrinkage-thrust away from masonry structures. When filled material is tamped or rammed after dumping, the ramming should be done by vertical blows."

The foregoing is quoted from printed specifications prepared by the writer, and is believed to be original and not to be found elsewhere. The specifications resulted from the writer's observations of the results of different ways of dumping and handling material, and such results are noted briefly as follows:

1.—A first-class masonry structure is usually ruined by being dumped against by a tail dump, the structure being forced out of plumb.

2.—A masonry structure, of inferior and light design and of poor material and workmanship, will give astonishingly good results if the filling is deposited back of it so as to flow away from it when dumped.

3.—The greater the shrinkage of the filling material and the higher the dump, the greater the force or pressure due to flow and shrinkage.

4.—The flow and shrinkage pressures are greatest in the longest layers of dumped material.

5.—The flow and shrinkage forces are in the direction of the flow of the material when dumped.

The results just described may easily be confirmed by observation. If an embankment 50 ft. high is made by dumping from both sides of a trestle—no matter whether the shrinkage of the material is small or great—it can be noticed that, after the embankment is completed, vertical cracks appear near the center line, and that frequently they are quite wide and extend downward a great many feet. If all the material filled back of an abutment is dumped so as to flow away from it, after the first soaking rain, a space can be found between the filling

Mr. Bonzano. and the abutment that varies in width approximately in proportion to the height of the dump and the coefficient of shrinkage of the material.

The writer doubts the value of the data secured by Mr. Goodrich, and feels confident that, for practical use, the data must be obtained by considering primarily the direction of flow when dumped, together with determining the coefficients of shrinkage. The writer could cite many interesting cases from which the foregoing conclusions were derived.

Mr. Hewes. V. H. HEWES, M. AM. SOC. C. E.—The author's description of the method used in obtaining the pressure has given the speaker an idea which it might be well to mention, so that someone may take it up, in future experiments, and elaborate it.

Taking into consideration the principle that carbon changes in electrical conductivity when placed under varying pressure, raises the point that, would it not be possible to place a carbon cylinder, or carbon in any shape, of a certain length, back of the retaining wall, and place the piece of carbon in circuit with a battery and galvanometer, and in that way get the deflections as the pressure varies on the piece of carbon? The conductivity being the reciprocal of its resistance, it would be more accurate and convenient to use a "Wheatstone bridge" to measure the variations of resistance due to changes of pressure. The carbon might be covered with a material which would prevent it from taking up moisture, which would cause a change in its resistance. Also, the piece of carbon might be placed in the mass of the material itself, and, in that way, the resistances could be obtained as the pressure on the carbon increased. The ends of the carbon could be exposed to the mass of the material, while the sides could be protected by a box, from which the ends would protrude, in that way causing the pressure to come on the ends of the carbon. The axis of the carbon could be placed in any direction in which the pressure was to be observed.

It might also be easy, in such cases, to have two or three sets of carbons placed in the mass of the material to be tested, and, by simply having a small switchboard, the experimenter could measure first one circuit and then another. The carbon might remain in for any length of time, and follow the changes from month to month.

It might be stated that, in using the block of carbon, all arching effects might be eliminated.

Mr. Dunham. H. F. DUNHAM, M. AM. SOC. C. E.—In describing curves of the variety under discussion, a positive check, or comparison with a pressure exactly known, would be valuable. If water were used as a material for "the fill," either in a cylinder or against planks, the precise pressure could be determined. The deflection could be compared with that observed for other material, or a curve for water under dif-

ferent heads could be compared with one for silt, quicksand or wet clays. It would be interesting to know whether this has been done. Mr. Dunham.

ROBERT BREWSTER STANTON, M. AM. SOC. C. E.—The subject of "Lateral Earth Pressures" has been one of great interest to the speaker for the past thirty years, especially in connection with the phenomena of great land slides, both natural and artificial, with which he has been specially connected, beginning with the construction of the Cincinnati Southern Railway in Tennessee in 1874. Mr. Stanton.

The author of the paper is to be congratulated upon his careful and painstaking study of the subject, from the technical and mathematical side of the question; and all engineers engaged in public works, where such conditions are encountered, owe him and other mathematical investigators a debt of gratitude.

Every engineer who has made a study of this subject, and has had occasion to apply the same in practice, knows full well the great value to the profession of all such scientific discussions, and the mathematical formulas deduced from experiments in the laboratory. The speaker has no intention whatever of casting any reflection upon such mathematical investigations, either pure or applied, for in years gone by he himself has been somewhat of a mathematical gymnast, but he desires to call attention to one or two very important facts, which, it would seem, are almost entirely omitted in investigating lateral earth pressures and land-slide phenomena.

More than thirty years' experience in contact with actual conditions in Nature, throughout the Rocky Mountain region from Canada to Mexico, has convinced the speaker that, in the construction of retaining walls, and other works to resist lateral earth pressures, and in the investigation of the causes of great land slides, two most important conditions are usually overlooked.

The mathematical formulas are absolutely correct, as deduced from the conditions, as stated; but, how often are just such conditions found on the mountain side, in the reservoir, or in the great land slides of the world? The mechanical conditions are there, and the mathematically calculated results should occur; but, how often are the beautiful formulas entirely upset and made utterly useless by Nature quietly inserting some little geological or chemical coefficient which has been entirely neglected in the calculations?

Hence, the point to which it is intended to draw particular attention, is the dependence of every such formula and all such investigations upon the "interdependence of all sciences, as applied to engineering works."

In the discussion of lateral earth pressures and the causes and effects of great natural land slides which produce such tremendous lateral pressures, two other sciences, namely, geology and chemistry, are absolutely necessary—in addition to the mathematics—and perhaps some others also.

Mr. Stanton. About eight years ago the speaker was employed by the Canadian Pacific Railway Company, to examine and report upon the great land slides on the Thompson River, in British Columbia, and it is intended now to give only a few facts from the results of that investigation, to illustrate the point in hand, for the reason that some, at least, may not find time to read a paper* by the speaker on this subject in which the peculiar conditions encountered and the methods of investigation are described at some length.

To be very brief: The slides referred to occurred at a point on the Canadian Pacific Railway about 200 miles east of Vancouver, B. C., near where the railway passes through the Black Cañon Tunnel. This portion of the Thompson River traverses a gorge about 5 miles wide at the top, and about 2 000 ft. deep, with hills and higher ranges rising back on each side to elevations between 5 000 and 7 000 ft. The river runs in an inner gorge. There is but little bottom land near the river, the real banks of which are from 50 to 150 ft. above low water, and which extend upward in benches and terraces back from the river and to much greater heights.

Up to the time of the speaker's examination it had been believed that the great slides were caused by a seam of clay on bed-rock, similar to the Portland slides, and so common in most such land slides or slips. The water from the irrigation of farming lands on the benches above (which had always been understood as the direct cause of the slides) was supposed to have sunk down through the underlying material, and, lubricating the stratum of clay on bed-rock, destroyed its friction, and enabled the mass to slip down an incline into the river, carrying the railroad embankments and cuts with it.

As an explanation, it may be of interest to state that in this section of British Columbia, only 100 miles east of the Pacific Coast, where the rainfall at Vancouver attains 62 ins., and at other points even exceeds 150 ins. per annum, there is a country as hot and dry as Arizona, where not a blade of grass can be raised without irrigation, and the temperature ranges from 120° in summer to —50° in winter.

After a careful and extended examination of the slides and the country around them, the speaker was convinced that no such stratum of clay and no bed-rock sloping toward the river existed, at least above the bed of the river, so as to cause the results noted; and that, although the irrigation water was in fact the direct cause of the movements, the question was not one of mechanics or mathematics, but rather one of geology and chemistry, influenced perhaps by some remoter sciences.

There were eight separate portions affected, six being comparatively small, with two very important slips known as the "North" and

* "The Great Land-Slides on the Canadian Pacific Railway in British Columbia." By Robert Brewster Stanton, M. Inst. C. E., *Minutes of Proceedings*, Inst. C. E., Vol. CXXXII, p. 1.

"South" slides. At both places the country originally sloped up from the river in a series of benches or terraces to the first line of hills. The south slide has an extreme length of 1 880 ft. along the railway, and an extreme width, back from the river, of 1 575 ft. It is of somewhat irregular form, with a semicircular outline at the back, and covers an area of 66 acres. The north slide has a maximum width at its widest portion of nearly 0.5 mile, and a length, back from the river, approaching 0.75 mile, with the same semicircular back line. It is of irregular form, and covers an area of 155 acres. The height of the first bench next to the river, in both cases, was originally about 80 ft. above low-water level. The land then rose in successive levels to a height, on the south slide, of 400 ft. to the bench at the top, or back edge, where the cave-down broke off the solid ground, and, in the case of the north slide, it extended to the third higher bench 500 ft. above the river. It is impossible to ascertain at what depth these enormous masses of earth and loose rock broke, or, in other words, the depth of the plane on which the mass moved toward the river; but it is estimated that at the back edge of the south slide the break fell almost vertically for a distance of more than 300 ft., and on the north slide perhaps more than 400 ft. Mr. Stanton.

Speaking generally, after the great valley of the Thompson River was cut out by glacial action, it was filled again with glacial drift and silt, and once more the river cut its channel down through this glacial deposit and now flows in a narrow gorge, within the greater one, which slopes in benches and terraces up to the hills and mountains on both sides.

The terraces on each side of the valley along this section consist of the soil on the top of each bench of light sandy loam to the depth of from 1 ft. to 8 ft. Below, in places, is found from 3 to 10 ft. of clean coarse river sand. Next occurs loose and nearly clean stratified gravel and boulders, and below this a partially cemented gravel with larger boulders. The material which holds together the gravel and stones of this formation is boulder clay, a porous arenaceous clay silt, through which water passes freely, yet which, in a dry state, will stand in vertical walls to a considerable height. It extends to a greater depth on the higher terraces; in places it is perhaps 500 ft. deep. The boulder clay is here found in two forms: in its original form, as first laid down; and, especially upon the lower benches next to the river, in a secondary or rearranged form. Under the lower benches, particularly under the slips, there is a deposit of silt or imperfect clay, which shows in places to a depth of from 50 to 200 ft. It is the same silt which forms and binds together the boulders of the boulder clay, but is entirely free from gravel or boulders. These lower deposits have been named the white silt deposits. "They are generally fine and uniform in texture, and are usually well bedded in perfectly

Mr. Stanton. horizontal layers from ¼ in. to 4 ins. in thickness," with occasional sandy seams and small pockets of coarse sand, formed locally, appearing in places.

By the continued application of large quantities of irrigation water upon the cultivated fields above, and upon the upper portions of what are now the slides, almost the entire surplus not absorbed by the plants or evaporated, sank down freely through the loose soil, sand and gravel; and, while not as readily, yet with considerable ease, through the boulder clay, and reached the underlying silt. After some years this water saturated the argillaceous silt and converted it into the form of river mud of about the consistency of thick pea-soup. Long before the whole mass, or even a very large part of it, reached a state of perfect saturation, the silt would lose its power of sustaining weight. In the two places here referred to, on account of the peculiar topographical and geological contour of the country, the water applied at the back was concentrated, comparatively speaking, into one channel of descent (in each place) to the body of silt below, and thence it penetrated in every direction. The process of saturation required many years to produce any results, for if a considerable quantity of the silt had become saturated to the point at which it would lose all cohesion, it would not move, on account of there being so great a distance to any point of outlet, together with the self-supporting power of the boulder clay in its confined position, which was nearly all absolutely dry over the slip; hence a large extent of the underlying silt became more or less saturated before it could find an outlet in any direction, even with a considerable weight upon it in its more or less semi-liquid state. Finally, when a large body of the silt had become saturated to such an extent that it could not sustain even its own weight, except in its confined position, and the limit of resistance, possibly in the form of an arch, of the boulder clay had been reached, the great mass of earth and boulders above—in the case of the south slide, estimated as weighing some 32 000 000 tons, and of the north slide approaching 100 000 000 tons—the whole mass dropped almost vertically, while the immense tracts of broken and mixed material seeking an outlet forced their lower sides out on the line of least resistance and found their way into the river. This action is distinctly shown by the almost vertical walls in the boulder clay along the outline of these two slides. While at their foot there is now a talus slope of crumbled material, these walls stand vertically to a height of from 50 to 200 ft., more clearly shown in the north slide, where the vertical cliffs of boulder clay, and in places of the silt itself, extend around the whole slide for a distance of more than 1.5 miles. It is also shown by the present position of large sections of the original surface of the highest bench, which broke off at the line of the back wall, and which now stand in the sunken mass at an angle of about 45°, with their

former level surfaces tilted back and away from the river. The back edge thus dropped first and lower than the portion some distance in front of it. In dropping and pushing out toward the river, the whole tract was broken into sections by great cracks, which still exist. The larger cracks run parallel with the river and at right angles to the line of movement, while other and smaller cracks run in every direction, cutting the whole into blocks of boulder clay and dry silt. Mr. Stanton.

In every instance noted, these slides occurred from 3 to 6 years after irrigation began at each point. In the case of the larger one, the great north slide, the final catastrophe was hastened by the bursting of a small reservoir. A very large quantity of water was necessary for raising crops, on account of the sandy nature of the soil and the nature of the subsoil. The topography of the several benches assisted materially toward the final result. Each field being in the form of a shallow basin, around which the irrigation ditches were built, little of the surplus water was drained off; hence the greater part of that not taken up by the plants or by evaporation ran toward the center of the field and soaked down in one channel.

A most important question here arose, and was pressed hard in the trial of a suit for injunction before the Supreme Court of British Columbia, which suit went through all the courts of appeal of Canada and was finally settled in favor of the Railroad Company in the Privy Council in London. The question was this: How could the silt, which melted so quickly into slimy mud in still water, stand in vertical walls 100 ft. high, and resist the action, for centuries, of the river running against its sides and not melt down at all?

As to the action of the water upon the peculiar masses of silt which at present underlie the benches and terraces along the Thompson River, a number of curious facts were noted in and around the south slide, which at first seemed most difficult to explain. The silt or imperfect clay, which lies at some points in this section in masses from 200 to 1 000 ft. in thickness, is generally fine and of uniform texture, and is usually well bedded in horizontal layers of from ¼ in. to 3 or 4 ins. thick. In its natural state it is hard and dry, like a soft sandstone, and, when held between the fingers and struck with a light hammer, rings like stone. A large piece of this silt, however, placed in a basin of water dissolves after a few minutes and falls down, not in a lump as clay, but mingles with the water, forming a semi-fluid mass like thick pea-soup. The same soft mixture was observed oozing out at many points along the foot of the slide, forced out by pressure from above; so the question arose, how was it that this silt stood in vertical walls from 10 to 100 ft. in height along the Thompson River, with the waters of the stream running along and against their base, and at high water some distance up them, and yet they had stood for ages, and were but little injured, except by slight atmospheric disintegration?

Mr. Stanton. The silt is formed of three principal parts—silica, in the form of coarse and fine sand, and alumina in two forms, first, disintegrated feldspar, simply separated mechanically into grains, both coarse and fine, also in the form of sand, and constituting a large part of the mass; and second, decomposed feldspar or plastic clay. Under the action of running water, the sand, both the silica and the disintegrated feldspar, is washed out of and off from the surface, while the decomposed feldspar is precipitated and forms a coating of true plastic clay upon the mass, which soon becomes impervious to the water and is practically indestructible, thus protecting the underlying silt from further action of the water. The result of a chemical examination * of the material is given in the paper referred to.

Mr. H. J. Warsap, the chemist who made the analysis, also suggests the chemical action and assistance of the carbonic acid of the atmosphere, and ammonia, if present in the clay and the lime, in forming this impervious coating. The mechanical action of separation and precipitation noted above, it is believed, accounts for all the peculiar phenomena observed at every point in and around the slides.

The great quantity of irrigation water soaked downward into the mass of silt. It would absorb 53% of water without changing its form, yet with only about 35% it would be incapable of sustaining any great weight except in its confined position. After the final breakdown, and its release into the river with the continual application of water, and still being under pressure, this semi-liquid silt, containing all its original constituents, is forced out at the foot of the slide in great quantities. If a man steps on this ooze he is likely to sink into it, while, within only a few feet of such a spot, when examined by the speaker, there lay a large block of the same silt which had fallen over into the river in a dry state, and over which the last season's high water had run; it stood up 3 or 4 ft. above the level of the river beach, and the speaker walked and jumped upon it without making any impression. Breaking off a piece of this block, it was found that, less than 1 in. under the surface, the silt was in its original form, and easily dissolved in water. In the river, under low water, were also observed great masses of this silt which had fallen over into the river in blocks, over the surface of which the river had run for years without carrying them away. On the other hand, the backwater in an eddy soaking through the cracks and getting behind and around other blocks dissolved them completely, and the river carried them away.

It has been suggested by Emil Kuichling, M. Am. Soc. C. E., that there may be other causes which enter into the formation of the protecting surface so quickly placed on the mass of silt or clay when exposed to the action of running water and which thus prevents its destruction. The particular additional science to which Mr. Kuich-

* *Minutes of Proceedings*, Inst. C. E., Vol. CXXXII, p. 20.

ling refers being that of biology. Mr. Kuichling has made quite extensive study and examination into the development of the vegetable, and by some thought to be even animal, life on the surface of newly exposed clay banks, especially in brick yards. Mr. Stanton.

It is not for the speaker to refute any such scientific suggestion when coming from Mr. Kuichling. The subject is one in which the speaker has been somewhat interested, and has observed and noted the growth of vegetable life, and its undoubted protection, upon almost vertical railroad cuts, along the Missouri River between Omaha and Platsmouth, Nebraska, where, from his personal knowledge, these banks have stood for more than twenty years, still showing the marks of the picks used in their first excavation, and also upon cuts on railroads and common wagon roads, in the neighborhood of Vicksburg, Mississippi.

The part that biology may play in the protection of all these banks or walls of clay or silt is a most interesting one, and one that should receive careful study, but it is not one that the speaker can now enter fully into, for two reasons: First, not having the necessary accurate and detailed information, and secondly, not having the time to give this subject sufficient study.

Still there are one or two considerations which would seem, without further knowledge, to answer partly, if not completely, Mr. Kuichling's suggestion when applied to the vertical walls of silt on the Thompson River.

The action of the running water, in taking away the silica and feldspar sand, and precipitating the plastic clay protection, is almost instantaneous; if it were not so, the wall would crumble and be washed away.

Is it possible that plant life, even though the seeds, howsoever minute and howsoever numerous, exist, as stated, in the clay itself, could germinate, take root, and form such protection in an inappreciable instant of time?

That such plant life does form and protect many clay banks, as observed and noted on the railway cuts in Nebraska and Mississippi, is entirely correct; but has not such formation required considerable time, as compared with the instantaneous action of the running water on the silt in British Columbia?

Further, from many experiments in the propagation of disease —the very subject which Mr. Kuichling was investigating for the New York Board of Health—it is claimed by very high authority that the germs from which the vegetable and even animal life come are more likely those deposited upon the clay from the air. Not attempting at this time to prove or disprove this claim, if it is true, it would seem that it would require a still longer time to produce such life and growth as would protect the silt walls from the water action.

Mr. Stanton. In either case, it would appear that the mechanical action of the running water, as described above, perhaps coupled with the chemical action, suggested by the chemist, Mr. Warsap, entirely accounts for the observed phenomena on the Thompson River.

However, the suggestion of geology, as a third science to be studied, places more emphasis upon the subject of this discussion—the absolute necessity for engineers engaged in the construction of permanent works, more particularly reservoirs, railway embankments, retaining walls and other structures intended to resist lateral earth pressure, to study, besides mathematics and mechanics, all kindred sciences which may possibly enter into and affect the final conclusions and results.

Hence, it is particularly urged upon young engineers, and in fact upon all, before constructing such works, to find out first exactly what the material really is, both geologically, chemically and otherwise, and then calculate mathematically its mechanical operation.

Perhaps it may not be out of place here to ask a question or two which, it is hoped, may bring out discussion which will throw light upon this general subject.

Has not the modern specializing of the engineering profession into the numerous special branches tended to cause many, particularly young, engineers, to feel—and, to their injury, to act upon the feeling—that their particular branch was the one all important, and thus be led into such errors as noted in connection with all previous investigations of the British Columbia land slides?

No one appreciates more fully than the speaker the advantages of special technical education and a thorough knowledge of one subject at least, and the disadvantages of its absence, and thus possibly being a "Jack of all trades," but did not the old-fashioned practice of civil engineering—which embraced everything except military works—impel the engineer, placed far out on the confines of civilization, and thrown upon his own resources, to look upon the problem in hand from more different sides, and not only through his own particular hobby, so to speak, and thus bring him nearer the composite truth, than the modern specialist, under the same circumstances?

These considerations are not advanced as dogmas or even positive opinions, but for the purpose and in the hope of drawing out from engineers, more competent to speak, their opinions on one side or the other.

Mr. Lamb. Richard Lamb, M. Am. Soc. C. E.—This subject is of great importance. Mr. Stanton seems to infer that the paper is more theoretical than practical. The speaker differs from him.

Those whose business is the designing of material-handling plants are aware of the paucity of empirical data in regard to lateral pres-

sures of various materials, and of practical conclusions deduced from such experiments. It would have been of benefit if Mr. Goodrich could have extended his experiments to a number of materials, such as coal, ashes and mineral ores, and given his deductions therefrom. Mr. Lamb.

In designing coal bins for use in New York City, the Building and Dock Departments require that in calculating the lateral stresses, 45° be used for the angle of sliding of bituminous, and 30° for anthracite coal. Tests made by the speaker indicate that dry bituminous coal will slide down an iron chute placed at 40° to the horizon, and down a wooden chute at 45 degrees. Anthracite will slide down iron at 30° and down wood at 35 degrees. In freezing weather, iron chutes often become inoperative at the angles above named.

In bins with flat bottoms and with side gates, the problem is changed; and the angle of repose due to friction of coal upon coal has to be considered, instead of coal upon wood or iron. Piles of bituminous coal, especially when lumpy, will stand almost vertically. In determining the lateral stress, as the vertical component is due to the weight of the coal contained between the vertical wall of the bin and the angle of repose of the coal, the lateral stress is inversely proportional to the angle of repose. If the Building and Dock Departments require the use of a lesser angle than the actual conditions demand, designers are compelled to provide a much more expensive structure than the so-called "practical builder" constructs successfully in other cities. The foregoing facts, together with the rules prescribed in the new Building Laws now in effect in New York City, impose a heavy and unnecessary burden upon the coal trade. A series of tests with coal, like those made by Mr. Goodrich, would permit of the establishment of standard data, which would justify the Building and Dock Department engineers in lessening their requirements.

The following formulas are used, by one of the leading coal-handling machinery firms, in calculating the tabulated statement, as shown in their catalogue, giving horizontal pressures of coal at various elevations, upon the sides of coal bins. These formulas were not deduced from experiment, but are based entirely on theoretical grounds. If there are any errors, they are on the side of safety, as many iron and wooden coal bins have been built, the calculations being made with these formulas. If experiments were made, undoubtedly the angles of slip would be increased and the bins made by the new formulas would be less expensive. With a coefficient of safety of 5, the extra strains imposed by frozen coal dropping, or an avalanche of coal, would be amply safeguarded.

PRESSURE EXERTED BY COAL AGAINST VERTICAL RETAINING WALLS, PER FOOT OF LENGTH.

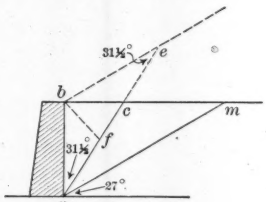
Angle of repose of hard coal = 27 degrees.
Weight of coal per cubic foot = 52.1 lb.

Mr. Lamb. *For Surface Horizontal.*—

$$\text{Vertical pressure} = \frac{\text{weight of the triangle, } a\,c\,b \times b\,c}{a\,b}$$

$$= \frac{52.1 \times a\,b \times b\,c \times b\,c}{2\,a\,b} = 26.05\ (b\ c)^2,$$

or, the pressure varies directly as the square of the depth, d.



When the Surface Slope is Equal to the Angle of Repose.—

$$\text{Vertical pressure} = \frac{\text{weight of the triangle, } a\,e\,b \times b\,c}{a\,b}$$

$$= \frac{52.1 \times b\,f \times a\,f \times b\,e}{a\,b} = 52.1\ (b\,f)^2,$$

again, the pressure varies directly as the square of the depth, d.

Hard Coal.—

Surface horizontal { Total pressure for depth, $d = 9.78\ d^2$. / Pressure on lowest foot $= 9.78\ (2\,d - 1)$.

Surface sloping { Total pressure for depth, $d = 14.22\ d^2$. / Pressure on lowest foot $= 14.22\ (2\,d - 1)$.

Soft Coal.—

Surface horizontal { Total pressure $= 6.37\ d^2$. / Pressure on lowest foot $= 6.37\ (d - 1)$.

Surface sloping { Total pressure $= 10\ d^2$. / Pressure on lowest foot $= 10\ (2\,d - 1)$.

Angle of repose = 35 degrees.

The speaker was recently engaged in designing a plant for handling the ashes and street sweepings of Brooklyn by trolley cars, and could find no data giving the angle of repose of ashes and street sweepings. He decided, by tests, that with iron chutes the angle should be not less than 30°, and designed a car with 34° for the angle of the floors. This car was not built, because it was learned that Col. Waring had a number of scows built with an angle of about 32° for the floors. They were expected to be self-discharging, but proved to be failures. Subsequently, the writer constructed some buckets, of about 7½ tons capacity, having trunnions and being designed to be dumped by derrick car and cableway. These buckets can be turned upside down. From operating these buckets, it has been observed that they will discharge the ashes when at an angle of about 35 degrees. On the

strength of this information, the writer is now designing a car with mechanism for discharging these large buckets from the cars direct. Mr. Lamb.

Ashes are often seen piled almost vertically. The lateral pressure of ashes, especially when bound together with rags, and other matter in the street sweepings, cannot be determined by any data available in the books of our profession, as far as the speaker has been able to learn.

Mr. Goodrich determined from his experiments the fact that the more finely a material is divided, the less the lateral pressure. He stated, however, that he considered that this deduction is of more theoretical than practical importance. On the contrary, the speaker wishes to emphasize the practical importance of this very deduction. Had this matter been brought to the speaker's attention before designing and building a large copper mining plant recently, he would have designed the ore bins differently. Steel floors, at an angle of 40°, were put in, and it was found that the crushed quartz ore, from 1½ in. to ⅛ in. in size, would slide down readily, but the very fine ore, up to ⅛ in. in size, would not move. A water jet, extending across the bin, had to be supplied in order to make the quartz fines move down a 40° steel-incline floor. It is evident, therefore, that the finer the ore is crushed, the less will be its lateral pressure when confined in a bin, which is what Mr. Goodrich proved by his experiments.

E. P. GOODRICH, JUN. AM. SOC. C. E. (by letter).—Subsequent to the preparation of this paper, but before the time of its presentation, two papers came to the writer's attention, both of which are of great interest in connection with the subject. Mr. Goodrich.

The interesting paper by D. D. Clarke, on "A Phenomenal Land Slide"* affords some data of a variety comparable with the work of the writer. The surface of the slip was practically a plane, and its slope a fairly uniform one, so that the tangent of the friction angle can be approximated with some precision. This was done, and the theoretical lateral pressure computed by Rankine's formula. The result was then plotted in Fig. 48, in connection with the curves observed in the cylinder experiments on clay. That slipping was actually stopped by draining off the water, and thus reducing the percentage of saturation of the clay from one approximating complete saturation to one of normal amount, proves that the friction angle, and hence the lateral pressure which the mass could have exerted, would lie between curves representing those respective percentages of saturation. Fig. 48 shows that the computed lateral pressure lies in exactly this relation to the observed pressure curves.

Mr. J. H. Jamieson, in his paper on "Grain Pressures in Deep Bins,"† shows that the curves of lateral pressures of wheat in bins

Transactions, Am. Soc. C. E., Vol. LIII, p. 322.

† Reprinted in *Engineering News*, Vol. LI, p. 236.

Mr. Goodrich. of different diameters are remarkably similar when plotted to scales which vary inversely as the diameters of the bins. Three of his sets of observations are thus shown in Fig. 49, where are also inserted the writer's observations on sand with the 3-ft. model, and those of steel for earth, with his 6-ft. bin. The two latter curves are plotted to scales related to each other and to Jamieson's work, as above described, but are also increased in proportion to the relative weights of wheat and earth (as 1 to 2). It is believed that the dissimilarity in these curves disproves Jamieson's theory of exact proportionality

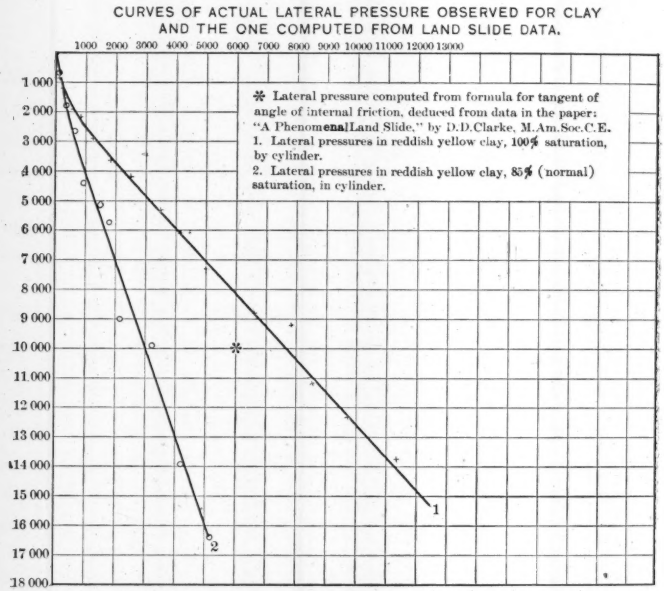


FIG. 48.

under such conditions, so that the theoretical value of his work is very much questioned.* This work does seem to prove conclusively at least the one fact, however, that models 12 ft. and less in diameter, produce friction and arch action; and, by inference, those retaining walls which are only that distance apart cannot be treated as if the retained material was of indefinite extent. Such walls will support more or less of the actual weight of the material retained, according to the depth, and unless a marked arch-action thrust be set up, will

*See also *Engineering News*, Vol. LI, p. 403.

be proportionally more stable than others, and can thus be designed with a smaller factor of safety, according to present practice in the use of that item. Mr. Goodrich.

In connection with the reconstruction of Dry Dock No. 2, at the New York Navy Yard, a set of observations was carried out upon the 12 by 12 in. yellow pine shoring timbers which braced the sheet-piling driven to retain the earth around the dock. P. J. Cleaver, Jun. Am. Soc. C. E., an Assistant Engineer in the Navy at the time, conducted the observations, and writes as follows concerning them:

"Chocks at the lower ends of several shores were crushed by the horizontal component of the stresses in the shores. On the basis of

REDUCED CURVES OF LATERAL PRESSURE OF SAND AND WHEAT IN BINS OF VARIOUS DIAMETERS.

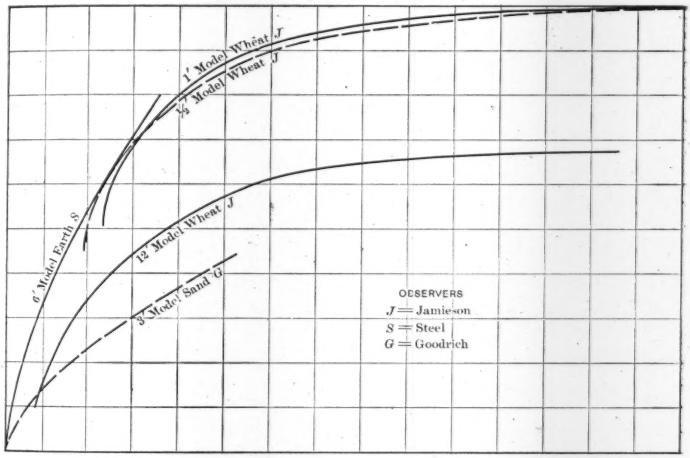


FIG. 49.

120 sq. in. for crushed area and a crushing strength for yellow pine across grain of 1 260 lb. per sq. in., the horizontal component of stress was 75.6 tons, making a compression in the shore of 84 tons.

"Several of the shores buckled, in one case 2 in. or a little more. This shows that the compression in the shores must have been considerably greater than their safe theoretical load. On the basis of $860 - 7 \times L \div H$ for safe unit load for long-leaf yellow pine columns, a safe load of 42 tons is indicated. As the factor of safety in the formula is large, it is considered that the shore would easily require 84 tons to buckle it 2 in. This is simply a check on the above observations.

"Taking 75.6 tons as the horizontal component of the stress in each shore, and as the shores were spaced 10 ft. apart, the pressure exerted against the top of the shore by each running foot of bank would be 7.6 tons."

Mr. Goodrich.

The shore rested against the bank 20 ft. above its base, and the earth was surcharged 15.5 ft. above the top of the shore. On the basis of 100 lb. as weight per cubic foot of earth, and the assumption of a uniformly increasing lateral earth pressure downward from the top of the shore, but starting with zero at the surface of the earth, a ratio of lateral to vertical pressure of 68% is indicated. For various reasons, the writer believes this to be excessive, but it only goes to prove the large size of the stresses indicated by many such observations.

The idea suggested by Mr. Bonzano, from his observations, and embodied in his specification, is in exact accord with the experiments of Darwin, who found a 25% greater pressure when the fill was made toward the plane of pressure than when made in the opposite direction. The writer, therefore, believes that his data are of more practical use than Mr. Bonzano suggests in the last paragraph of his discussion.

The suggestion of Mr. Hewes concerning the use of carbon is ingenious, but the writer's experience with the cylinder, showed that even that mechanism was almost too sensitive, and that the size of the plug (1 in. in diameter) was altogether too small except for materials with grains less than ¼ in. in diameter. The carbon cylinder, however, might be used in connection with two opposing discs, somewhat as Wilson arranged his apparatus.

In direct line with the method suggested by Mr. Dunham, the writer endeavored to make a preliminary test of the cylinder, with water as the material tested. The fit of the plug and of the disc was not sufficiently tight, however, to afford useful results. In the experiments carried out by Mr. Jamieson, referred to above, use was made of a device in which the idea was to equilibrate the grain pressure directly by water pressure against the opposite sides of the diaphragm of the testing mechanism. The reliability of the results obtained, even with this arrangement, has been questioned by some.*

Mr. Stanton is right in contending that a knowledge of the mathematics and perhaps the physics of earth pressures is not always adequate. The chemistry of the subject is also necessary, especially when dealing with most clays and many mineral combinations. In this connection special reference must be made to the investigations of the United States Geological Survey of the chemical properties of clay, published in Professional Paper No. 11, "The Clays of the United States East of the Mississippi River," by Heinrich Ries, pp. 27–30.

It is to be noted, further, in connection with Mr. Stanton's observations, that ordinary blue clay of normal humidity will commence to creep under loads of about 12 000 lb. per sq. ft., corresponding to

* See *Engineering News*, Vol. LI, p. 403.

depths of about 120 ft., while movement is relatively rapid at double this pressure. The thickness of the masses described by Mr. Stanton were 300 or 400 ft. and more, and must have been very near the danger point even before the irrigation work commenced, unless the mass was almost entirely devoid of moisture. Mr. Goodrich.

The writer desired very much to include coal in his series of observations, but the results obtained with the gravel showed that observations with the apparatus at hand would have been worthless. It is hoped that in the near future some one will do some extended work along that line. The friction angles given by Mr. Lamb are all those of the friction between the material and steel or wood, and not properly those of either internal or surface friction of the material itself. The tangent of the angle of internal friction for cinders, as found by the writer, was 0.474, and of surface slope was 0.86, while Mr. Lamb gives 35° (tan. 0.7) for the angle of sliding of cinders on iron. His experience with fine and coarse copper ore, however, shows that it obeys the same general law when sliding on steel as when sliding on itself, which is an unexpected but remarkable confirmation of the results obtained by the writer.

Acknowledgment is herewith made by the writer of his appreciation of the kind remarks made by those discussing his work.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 984.

A PHENOMENAL LAND SLIDE.*

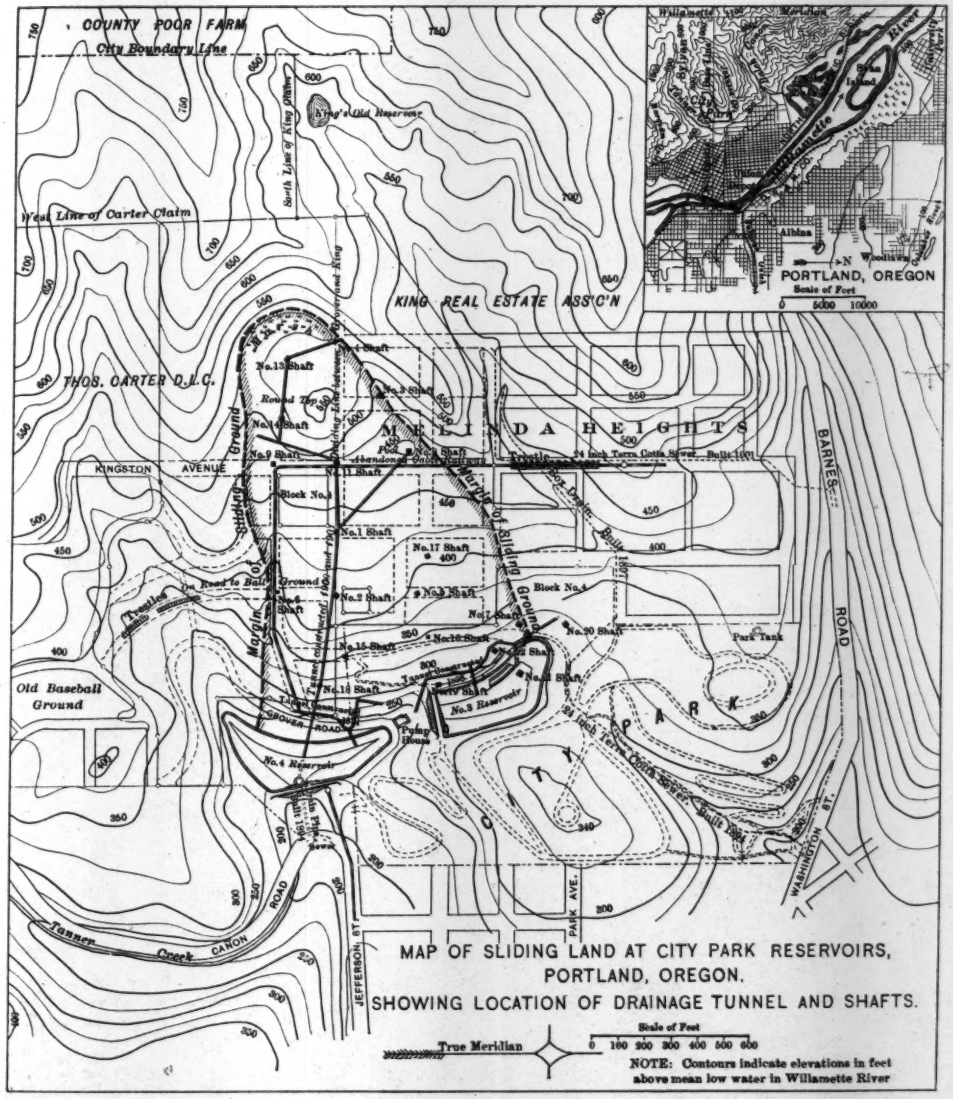
By D. D. Clarke, M. Am. Soc. C. E.

With Discussion by Messrs. George L. Dillman, Arthur L. Adams, James D. Schuyler and D. D. Clarke.

The gravity system of water-works, constructed during the years 1893 and 1894 by the Water Committee of the City of Portland, Oregon, included a series of four reservoirs for supplying the different districts of the city. Reservoir No. 1, with a capacity of 12 000 000 galls., and No. 2, with a capacity of 20 500 000 galls., supply the East Side District; and Reservoir No. 3, with a capacity of 16 400 000 galls., and No. 4, with a capacity of 17 700 000 galls., supply the West Side District. Two of these reservoirs, Nos. 3 and 4, on the West Side, were built in a narrow ravine occupying a portion of the City Park, about two miles west of the business center of the city. The lower reservoir has an elevation of 220 ft. and the higher one 290 ft. above mean low-water level of the Willamette River. These two reservoirs were formed by dressing down the banks of the ravine in which they are located; and, since their completion, a serious derangement of the western slopes of both reservoirs has taken place, owing to a movement of the adjacent hillside.

The magnitude of this movement was not understood at first, but it has since been found to extend for nearly the full length of both

* Presented at the meeting of April 6th, 1904.



MAP OF SLIDING LAND AT CITY PARK RESERVOIRS,
PORTLAND, OREGON.
SHOWING LOCATION OF DRAINAGE TUNNEL AND SHAFTS.

True Meridian

Scale of Feet
0 100 200 300 400 500 600

NOTE: Contours indicate elevations in feet above mean low water in Willamette River

reservoirs, a distance of about 1 100 ft. This distance may be called the approximate width of the slide at its lower or eastern end. At the apex or western end it is about 400 ft. wide, and its length from east to west is approximately 1 700 ft. The depth, as determined at various places, ranges from 50 to 112 ft., and the surface area approximates 30 acres. The volume of the moving mass has been estimated at 3 400 000 cu. yds.

Taking into consideration the characteristic features of this movement, its length, breadth and depth, and the uniformity of the movement of the sliding mass, it may be truthfully called "A Phenomenal Land Slide." It is the purpose of this paper to describe the surveys and explorations which have been made during the last nine years, for the purpose of determining the dimensions of this slide and its probable cause, and to aid as well in devising a plan for the cure of the difficulty.

The writer has been engaged in the service of the Water Committee since March, 1893, and for the past seven years has had engineering charge of all construction work. He has therefore had personal knowledge of the work from almost the very beginning, and the surveys and explorations described in the paper have been made largely under his personal supervision.

In order to reach a correct understanding of the situation of these reservoirs, and the reasons which led to their location at the point named, a few words regarding the physical characteristics of the region may be necessary. The City of Portland is largely built on the lower slopes of a range of hills bordering the Willamette River on the west. For a distance of between one and one and a half miles from the business district the slope is quite gradual, rising in that distance to an elevation of from 150 to 250 ft. Beyond that point the hills rise more abruptly and reach an elevation of from 800 to 1 000 to 1 200 ft. in a total distance from the river of three miles, or less. See Plate XVII, a map of the territory.

Flowing down the eastern slope of this range of hills there are several small streams which discharge into the river within the city limits through brick conduits of large size. These streams, though small, have furrowed out channels for themselves, which, on their upper courses, vary in depth from 40 to 200 ft. below the general level of the adjacent ridges. One of the largest of these streams is known as Tanner Creek, its lower portion being now confined in a brick sewer,

approximately 6 ft. in diameter. The upper or western end of this sewer is near the southeast corner of the City Park, and only a few hundred feet from the site of Reservoirs Nos. 3 and 4, to which this account refers. Above the head of this sewer the valley of the south or main branch of Tanner Creek is about 150 ft. wide, with side slopes from 1 on 1 to 1 on 3 or 4 horizontal, and with almost perpendicular bluffs of basaltic rock at a few points.

The reservoirs described are upon the north branch of Tanner Creek, which flows from the northwest through a corner of the City Park and unites with the main creek at the head of the brick sewer to which reference has been made. The total length of this branch is somewhat more than one mile, and it has a total fall of about 500 ft. During the dry season the flow is insignificant, and, for several months, it disappears almost entirely.

The selection of this ravine, for the site of the two reservoirs required for the west side of the river, was due chiefly to its favorable location for securing the desired elevation at the most accessible point for making connection with the system of pipes then in use. Besides, the land for one reservoir was already owned by the city, and the additional ground, needed for the second or low-service reservoir, adjoining the City Park and combined with it, could be purchased upon favorable terms.

There are other ravines of a similar character both north and south of Tanner Creek, but none more accessible or apparently more favorable as a reservoir site. The location of the reservoirs upon level ground, or outside of some ravine, was not regarded as feasible, within the required limits as to distance and elevation. At the point chosen for the reservoirs, the original bed of the ravine was quite narrow, from 20 to 50 ft., with sides sloping back about 1 on $1\frac{1}{2}$ to $2\frac{1}{2}$ horizontal, to a height of about 50 ft. on the east, and on the west about 100 ft. above the bottom.

The material composing the hillside, as far as appeared from the borings and examinations made before the excavation of the reservoirs was undertaken, was yellowish sandy clay upon the surface, but only a few feet in depth, with basaltic rock underneath. The character of the underlying rock was not then known, further than what could be seen at several points where a bluff a few feet high showed rock at the surface, and at one point, 200 yds. down stream, where a perpendicular bluff 50 ft. high was exposed. From these indications the general

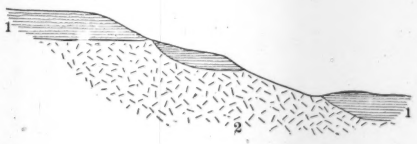
basaltic character of the rock was known. Where exposed, the rock was full of seams, but appeared to be sound and in place.

Along the banks of Johnson Creek, which flows in the next ravine north of Tanner Creek, about one-third of a mile from Reservoirs Nos. 3 and 4, similar bluffs of basalt can be seen, a road-metal quarry having been opened at one point exposing a vertical wall, from 50 to 75 ft. high, which shows plainly the general character of the material.

Here it may be of interest to note the following account of the geological features of this region, quoted from "A Geological Reconnoissance in Northwestern Oregon," by Joseph Silas Diller.*

"If the whole of western Oregon subsided 200 ft. in all parts, so as to restore at least in some measure the conditions of land and sea which obtained during the Pleistocene epoch, it is evident that the sea would flow in over the land, making a large bay of the Columbia and extending up the valley of the Willamette as far as Salem" (50 miles south from Portland). "The Pleistocene water body in its general outlines must have resembled Puget Sound, and to designate it specifically, as already indicated, Professor Condon called it Willamette Sound. The fertility of the Willamette Valley is largely due to the sediments deposited in it during the time it was a sound, and some of the plains and prairies may then have been formed.

"The data for the accurate determination of the depth of the water in Willamette Valley have not yet been fully made out, but the evidence already known to Professor Condon indicates that the water extended as far south as Spencers Butte, three miles from Eugene" (125 miles from Portland). "Judging from the height of the terraces on the Columbia, near the mouth of the Des Chutes" (90 miles east from Portland), "he estimated the depth of the water over the place where the city of Portland now stands to have been 325 ft. This may well be, and yet when we study the deposits of which the hills about Portland are composed, a much greater depth of water is indicated. During a brief stay in Portland a rough section was made up the slopes of the ravine from Gambrinus" (or Johnson Creek). "It is illustrated by Fig. 15. The city is



SECTION OF PORTLAND HEIGHTS, NEAR GAMBRINUS RAVINE.

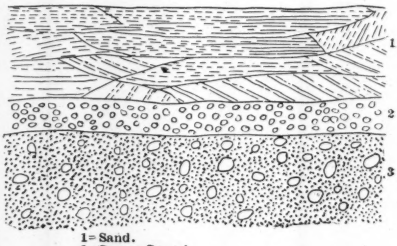
1 = Clay.
2 = Basalt.

"FIG. 15."

* Seventeenth Annual Report, United States Geological Survey, 1895-1896, Part 1. pp. 485 and 486.

largely upon the modern flood plane of the river, and is, at least at a number of points—for example, on Washington Street, near the Oregonian Building—made up chiefly of clay. The lower portion of the hill, above the general level of the city, is made up of basalt, with occasional masses of fine sediments, showing traces of stratification. The upper portion of the hill, extending from the top of the lava, at an elevation of about 310 ft., to the general plain above, which is at an elevation of nearly 660 ft. above the sea, is fine, argillaceous sediment closely resembling the loess of the Mississippi Valley. It is in places distinctly stratified and was evidently laid down under water. If this material was deposited in the Willamette Sound of Condon, as appears to be the case, the depth of water at Portland must have been not less than 600 ft. At present too little is known of the geology of the Portland region to assert that the fine sediments on the heights immediately west of Portland were deposited at the same time as those along the coast.

"At the north end of East Portland, near Albino, a bluff exposes the section shown in Fig. 16.



"FIG. 16."

"The 40 ft. of coarse sand above is well stratified, but irregular and cross bedded. This, with the 4 to 10 ft. of conglomerate next below, indicates strong, shifting currents. The lower 30 ft. of the exposure is made up of sand, pebbles and bowlders irregularly intermingled. At the base of the cliff, and occasionally within it, are found bowlders of soft gray or yellowish sandstone, very like the Tertiary sandstone exposed at various places in western Oregon. No fossils were found at this point, but Dr. David Raffety gave me a fragment, collected from the gravel at Brooklyn Mills, that contains *Arca microdenta* Conr., a common Miocene form. Brooklyn Mills is at the south end of East Portland. The bluff, in general composition and position, is practically a continuation of the one at Albino. The fossils found at Brooklyn Mills are apparently in a small bowlder derived from the Miocene, and indicate that the gravels in which the bowlders occur are of later age than the Miocene. They are doubtless Pleistocene, and probably younger than the high-level sediments on the opposite side of the river. The fossiliferous Miocene in place is not known to the writer nearer Portland than the Scappoose, in Columbia

FIG. 1.—LOOKING SOUTH FROM RIDGE NEAR CABLE RAILWAY BRIDGE, SHOWING RESERVOIRS NOS. 3 AND 4, PART OF SLIDING LAND AND SHAFT NO. 1.

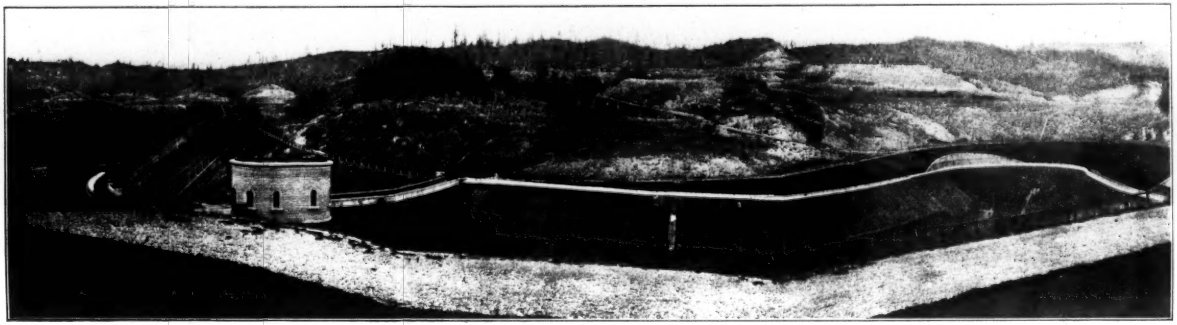


FIG. 2.—LOOKING WEST FROM CITY PARK, SHOWING RESERVOIR NO. 3 AND THE SLIDING LAND. PHOTOGRAPHS TAKEN SEPTEMBER 22D, 1897.

County, or Dilley, in Washington County, about 25 miles away. It is probable, however, that the same series of strata occur at no great distance south of Portland, and in the Willamette Valley and in the hills to the westward, and, furthermore, they probably extend beneath the city, where they are covered by later deposits."

In the foregoing extract, it will be noted that Professor Diller comments on the absence of fossils in this vicinity, as far as at present known, from which the age of the formation could be determined. As bearing upon this point, the writer would here mention a fossil which was found, on December 31st, 1898, in one of the excavations at the site of Reservoir No. 3, the upper reservoir at the City Park, at an approximate elevation of 218 ft. above the sea level. The fossil was found embedded in a deposit of blue, sandy clay, 41 ft. below the surface and 17 ft. above bed-rock, in connection with some small pieces of wood and a few water-worn pebbles.

Concerning the classification of this fossil, Mr. F. A. Lucas, of the National Museum, Washington, D. C., writes as follows:

"It is the left lower molar of a camel. Its worn condition renders it very difficult to identify, but it is probably *Camelops kansanus* Leidy. This species has been ascribed by both Leidy and Cope to the Pliocene, while Wortman reports it from the Pleistocene."

During the fall of 1891 and the following winter, or nearly two years prior to the commencement of reservoir construction, a cable railway was built from the business portion of the city westerly to the southeast corner of the City Park. Thence it continued its westerly course, crossing the ravine in which the reservoirs are now located and ascending the steep slope of the ridge for a distance of about 1 500 ft., and thence, turning to the north along Kingston Avenue, it followed a nearly level grade for about 1 200 ft. to the terminus of the line. In its last course along Kingston Avenue the road crossed two ravines, the first upon a 25-ft. embankment, and the second upon a timber trestle, 40 ft. high and 300 ft. long, the latter being across the main ravine in which the reservoirs are situated.

The photographs, Fig. 1, Plate XVIII and Fig. 1, Plate XIX, show the reservoirs and the sloping hillside on the west from different points of view, the cable road and the bridge across King ravine, to which reference has already been made.

The view shown in Fig. 1, Plate XVIII, was taken on September 28th, 1897, looking south from the point of the ridge northeast from the

cable railway bridge on Kingston Avenue. At the left can be seen the gate-houses at Reservoirs Nos. 3 and 4 and the valley of the south branch of Tanner Creek. In the center appears the terraced ground at the left of which can be seen the cluster of small firs marking the southwest corner of City Park. In the center of the terraces can be seen the temporary pump-house marking the site of Shaft No. 1. On the right can be seen the railway bridge, and, farther on, the outline of the "round top," near the head of the sliding ground, can be traced.

The photograph shown in Fig. 1, Plate XIX, was taken on September 22d, 1897, looking north along the cable-railway track on Kingston Avenue from a point on a ridge south of the moving ground. The 90° angle in the track is shown in the foreground. A short distance beyond the center is the twisted track at the northern margin of the moving ground. Kings Heights, north of Johnson Creek, appear in the distance.

The photograph, Fig. 2, Plate XVIII, was taken on September 22d, 1897, looking to the west from a point in the City Park, about 160 ft. northeast from Gate-house No. 3. On the left can be seen a portion of Reservoir No. 4, with the rip-rap on the west slope above the roadway. In the center can be seen the gate-house and a portion of the dam and basin of Reservoir No. 3, with the graded slope extending to the Park boundary. The clump of trees at the left of the center is at the southwest corner of the City Park.

The highest point of the ridge shown near the center is approximately 2300 ft. distant, and 400 ft. above Reservoir No. 3.

On the right can be seen a portion of the buttress built against the west slope of Reservoir No. 3.

The photograph, Fig. 2, Plate XIX, looking west, shows the angle in the cable-railway track just east of Shaft No. 6. The pressure of the slide had warped the track out of position, the rails having been bent to the south. When this movement was first observed, September 25th, 1895, the lateral movement of the rail amounted to about 6 ins. As shown in the photograph, the rail is 18 ins. from the original position, measured at an angle of 60° from the horizontal. A similar movement of the north track had taken place, but it does not show in the photograph. Toward the top of the picture a vertical bend in the south rail of the south track can be distinguished. This point is at the east side of the "ball ground" landing, where there is a break in

FIG. 1.—CABLE RAILWAY ON KINGSTON AVENUE.
SEPTEMBER 22D, 1897.

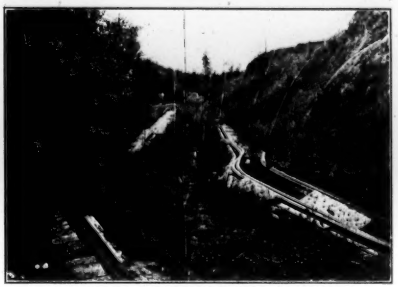


FIG. 2.—CABLE RAILWAY EAST OF SHAFT NO. 6.
JUNE 24TH, 1898.

NO. 3.—LOOKING NORTH FROM THE SUMMIT OF THE RIDGE, NEAR THE OLD BALL GROUND, SHOWING THE GATE-HOUSE, POWER-HOUSE AND PARTS OF RESERVOIRS NOS. 3 AND 4.
PHOTOGRAPH TAKEN SEPTEMBER 28TH, 1897.

the grade. It has been observed that all rail joints along this section of the track are tightly closed.

The view shown in Fig. 3, Plate XIX, was taken on September 28th, 1897, looking north from the summit of the ridge near the old ball ground south of the reservoirs. On the right can be seen the dam, gate-house and about half of Reservoir No. 4, and also the power-house and the gate-house and principal part of Reservoir No. 3. To the west of the power-house and Reservoir No. 4 is shown the roadway and the rip-rapped slope above it, and beyond, and farther to the left, is the small grove at the southwest corner of the City Park. Still farther west can be seen the terraced ground, with the temporary pump-shed at Shaft No. 1 standing near the center.

The cable road was completed about May, 1892, and put into operation at once. During the construction of the road, the owners of the property west of the City Park made contracts for grading and terracing their property, with a view of offering their lots for sale, the cable road having been extended to this tract solely for the purpose of making this property accessible and desirable as a residence district.

As has been stated, the operation of the cable road was begun in May, 1892, but the running of cars was discontinued during the following winter. The operation of the road was again commenced about May, 1893, and continued until September, 1893, when it was finally suspended owing to the removal of the bridge across the ravine at the site chosen for Reservoir No. 4. A change was made necessary by the beginning of work on the reservoirs, and the owners of the property preferred to abandon the line rather than reconstruct the bridge to span the reservoir, and protect it properly, in accordance with the terms of their right-of-way agreement.

During the grading of the property west of the reservoirs, to which reference has already been made, ridges were cut down and several small ravines filled, without providing proper underdrainage, and, in the two years following, other ravines were filled with materials furnished to the property owners by the contractors who were excavating the reservoirs. In the bottom of two of the larger of these ravines, rough log culverts were built, but, evidently, little or no care was taken to prevent the earth from sifting through the chinks between the logs, and the drains soon became choked and useless.

At one point in the ravine west of the cable road the embankment formed a shallow pool. For the purpose of draining this pool, as well as street intersections lower down the slope, the property owners constructed a terra cotta pipe sewer, some 8 or 10 ins. in diameter, laid a few feet below the surface, and leading eastward to the park boundary.

The principal part of the work just outlined was completed before the excavation for the reservoirs was begun. The excavation for both reservoirs was commenced in October, 1893, and carried forward simultaneously at both points during the following winter, but the work was not entirely completed until September, 1894.

During the winter of 1893–94 municipal affairs occupied the attention of a large committee of citizens, styled "The Committee of One Hundred," and during this time the new water-works, then in process of construction, came in for a share of the criticism to which all branches of the City Government were being subjected. The water-works being entirely under the control of "The Water Committee," consisting of fifteen of the leading and substantial men of the city—named for the position in the legislative act authorizing the work—their management was not to be impeached, but the engineers of the Water Committee were criticised by some members of the Committee of One Hundred for recommending the location of important reservoirs in the City Park. As a result of these criticisms, the Water Committee called upon their engineers for a report on the condition of the work. In compliance with this request, the engineers submitted the following statement:

"On account of the elevation of the headworks on Bull Run, the fall required to overcome the friction of the water in the pipe and the allowable pressure on the city mains and the submerged pipe under the Willamette River, the reservoir must be placed at an elevation of about 300 ft. above the base of city grades.

"By survey made from the southern to the northern boundary of the city, it was ascertained that all the lands at this elevation were on a steep hillside; that the reservoir could only be constructed in ravines in which the required capacity could be obtained by dams of moderate height, and the depression in the City Park was best suited for the purpose of a reservoir, and was the only one into which the water could be discharged without encountering great and almost insuperable difficulties in the extension of the supply main from the crossing of the Willamette River westward. From borings and test pits made on the

side slopes of the ravine, it was ascertained that the material was a light surface soil, underlaid with clay and hardpan resting on solid rock, as shown on plans and sections recently laid before the Water Committee and Messrs. Woodward, Honeyman and Foley of the Committee of One Hundred.

"The dam proposed will rest on solid rock at the sides and bottom, and the lining of the sides on clay, hardpan or rock, for which purpose all the loose surface will be removed. Around the reservoirs, about 5 ft. above the surface of the water, there will be a berm 10 ft. in width, along which will be laid culverts and drains to carry away the water running from adjacent lands.

"It was stated by Mr. Woodward, in conference with the Committee on Monday last, that he did not apprehend danger from failure of the dam or leaks from the reservoir, but that the lands of the King Real Estate Company sloping downward to the west line of the City Park, and resting on a bed of clay, underlaid by rock, that this clay when exposed to the action of water would become soft and slippery, so that the whole hillside would slide down into the upper reservoir, and be dumped into the lower, causing the destruction of both and loss of life and property from the discharge of water contained in the reservoirs. He also stated that should there be a dip in the strata of rock at the foot of the hill and a rise toward the reservoir the danger of a slide would be obviated.

"The construction of roads and terraces on the lands of the King Real Estate Company, and the wash of small streams caused by recent heavy rains, furnish ample data to observe the nature of the soil and the underlying strata of clay and rock. From the top to the bottom of the hill, the rock crops out in places, indicating that there is no great depth to the surface soil. At the points where the rock is exposed there are no indications of a slippery subsoil or tendency to slide. The slopes are not deep, and extend, not to the reservoirs, but to a depression at the foot of the hill, on the west line of the City Park, along which there is a road leading to the bridge in the City Park below the reservoir. On the western edge of the reservoir there is another road, and between the two there is a knoll extending 400 ft. north of the dam. Looking from the east side of the reservoir toward the deer park, the land appears to rise continuously to the top of King's Hill, but, as above stated, there is a road and depression on the other side of the knoll, from which there is a gentle slope to the top of the hill.

"Within the deer park the depth of the surface is not known, but the rock crops out on the east side and west side, and to the north, and the cost would not be great to strip the rock so there would be no danger of a slide or damage to a reservoir should a slide occur. To the north of the deer park and on the land of the King Real Estate

Company, there had been a surface slide extending down to the upper edge of the reservoir, but this slide, as well as the others which have occurred at both reservoirs, can be traced to the action of a small stream of water which has run over the surface soil and saturated it to the clay on which it rested. This, and all the others, are small and local, and can be easily remedied.

"The side slopes of the reservoir have not yet been cut down to firm materials, and the wash of the surface soil by the heavy rains of last winter gives to the excavation a very rough appearance; but there have been no large displacements or slides, and, in our opinion, there is no danger in the future of any slide of sufficient magnitude to injure the reservoirs."

The bridge referred to in the foregoing report crossed the ravine immediately in front of Dam No. 3, affording access to the elk barn and enclosure which formerly occupied the summit of a small knoll 200 ft. from the west end of the dam.

The depression near the west line of the City Park was filled, during the reservoir construction, with materials excavated from the basin of Reservoir No. 3. See Fig. 2, Plate XVIII.

The publication of this report seemed to quiet the fears which had been aroused by the statements made before the Committee of One Hundred, and public interest in the matter soon began to wane. The final report of the Committee of One Hundred, submitted to the citizens several months later, contained no reference to the reservoir investigation.

In the light of subsequent developments, the foregoing report of the engineers of the Water Committee appears to indicate a failure to comprehend the magnitude of the difficulties with which they had to contend, but, for such failure, the writer has no words of criticism to offer. He was at that time a member of the engineering staff, and, although indirectly connected with the work in question, he was acquainted with the circumstances of the case, as then understood, and was in entire accord with the position taken by the engineers in their report.

The engineers state in their report that it was claimed by some that the hills west of the reservoirs were of clay resting on rock; that the clay would slip and slide when wet; and that, in consequence of such slides, the reservoirs would be suddenly filled and the water they contained spilled out, causing great loss of life and property.

Presumably this opinion was based upon what could be seen upon

the surface near the reservoir, in addition to what was known of the material at points in other ravines where slides had occurred in former years. That no person at that time had any conception of the existence of a deep-seated movement, of the magnitude that has since been developed, is made evident by the fact that they predicted surface slides which would suddenly fill the reservoirs and cause death and loss of property by the flooding of the low ground below the dams.

In examining the ground adjacent to the reservoirs, the engineers found only indications of small surface slips. These they thought could be rectified, and the ground made stable at small expense, and they therefore reported in favor of continuing the work. It can now be seen that the engineers were mistaken in their judgment, and failed to realize the magnitude of the forces arrayed against them. The slide, which was even then doing its destructive work, was not near the surface of the ground, and hence not apparent to the ordinary observer. Even those who had early knowledge of a movement upon different portions of the "sliding land," as it is called, failed to connect and unite them into one harmonious whole, and so far failed to estimate properly the extent of the movement which was taking place.

The surveys and borings which determined the selection of the City Park as the site for the reservoirs were made chiefly during the year 1887, supplemented, however, by additional borings made early in the year 1893.

The work of reservoir construction was pushed rapidly during the spring and summer of 1894, grading for reservoir basins, work upon concrete linings and the massive concrete dams for both reservoirs being in progress at the same time. The excavations for the reservoir basins were not completed until about September 1st, 1894.

In the early part of August, and before the west slope of Reservoir No. 4 had been cut down to its intended position, a slip in the bank was noted about midway on the slope opposite a point where the basin was about 30 ft. deep. (This point is marked by a cross in Fig. 2, Plate XX.) The slopes of the reservoir basin were 1 on $1\frac{1}{2}$.

The line of this slip was found to be along a seam between strata of blue and yellow clay, the yellow clay being above the blue. The slope of the seam was found to be 1 vertical and 10 horizontal, the dip being westward into the hill.

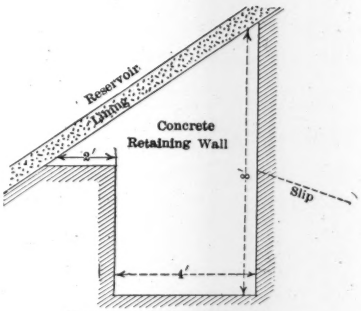
Concerning the character of the deposits of blue and red or yellow clay, which were uncovered at this point, and which have figured largely in all subsequent explorations, the following may be said: The deep yellow and red clays found at different points are, as a rule, quite plastic and contain little sand. Evidently, they were formed by the decomposition of the lava, which at different periods has overflowed this region, and were colored by the iron in the rock. Some beds were found where this process of decomposition seems not to have been complete, for the texture of the material was coarse, containing grains and small fragments of the rock incorporated with the clay, but the finer materials were all of the same reddish or yellow color. The blue clay differed from the red in some of its characteristics. When found in thin seams near bed-rock, it was tough and plastic, but when found in considerable bodies at higher levels it contained quite an appreciable portion of fine sand. When dry it was hard to pick, and a vertical bank would stand without support. When placed in water it soon crumbled into an incohesive mass. The general characteristics of this clay indicate its sedimentary origin, in some deposits small pieces of wood and water-worn pebbles being found in connection with it. In his later treatment of this material, the writer has described it as "blue quicksandy clay" as best indicating its character.

Owing to the loose material on the face of the untrimmed bank at Reservoir No. 4, the extent of the slip, referred to above, was not determined for several days, and it was thought to be merely a local slip which extended but a few feet above the berm at the top of the reservoir slope. In fact, there was, at about this time, and near the same point, a local slip extending a few feet into the bank, which was subsequently refilled and a part of the parapet wall and slope lining built over it.

After a few days' observation of the movement of this slide, it was found that the break in the ground extended about 200 ft. south and 100 ft. north from the point where it first appeared, making a total length along the reservoir slope of approximately 300 ft. When first observed, and for several days thereafter, the movement was at the rate of $\frac{1}{2}$ in. per day, but this did not continue long.

The plan adopted to overcome this difficulty was to build a concrete retaining wall in front of the slip and below the reservoir lin-

ing. This wall was of the dimensions shown in Fig. 1, for a total length of about 320 ft.



CONCRETE RETAINING WALL, BELOW RESERVOIR LINING AT "SLIP"

FIG. 1.

At the time this wall was being built, a drainage tunnel was constructed in the rear of the wall, following as closely as possible the seam dividing the blue clay from a deposit of yellow clay with a mixture of loose rock, which was found to be water-bearing. The tunnel extended the full length of the buttress, and from 50 to 100 ft. westward therefrom. Several small pockets of water were tapped and drained into the sewer through pipes laid in this tunnel during its construction.

After the completion of this wall it was observed carefully for some days, and, no further movement appearing, it was thought that the retaining wall and drainage tunnel acting in conjunction had been effective in checking the movement. Accordingly, the lining of the west slope was completed and the reservoir made ready for use in accordance with the original design. The filling of the reservoir was completed on December 17th, 1894.

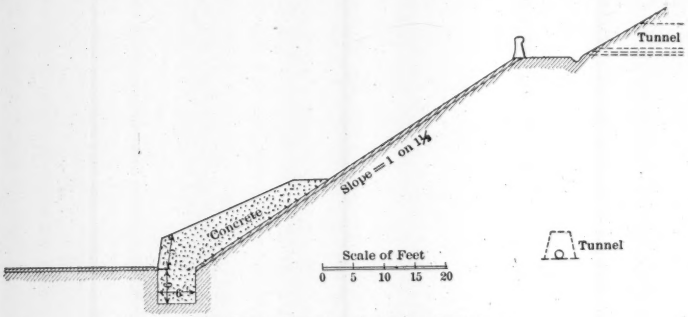
The work at Reservoir No. 3, the upper or high-service reservoir, will be described next.

Early in September, 1894, the work on this reservoir had progressed so far that the lining had been completed on the bottom and the west slope, when the discovery was made that the concrete lining in the bottom had bulged up at one point, just north of the center, from some cause then unknown. A few days later, September 8th, this break was repaired, but before the end of the month another one was discovered in the bottom, near the foot of the west slope and opposite the former break. This time the difficulty was thought to be due to quicksand and clay in the bank behind the facing of more stable materials upon which the slope lining had been laid.

To meet the difficulty encountered at this point it was decided to construct a concrete buttress wall about 100 ft. in length, and of the general dimensions shown in Fig. 2.

At the same time that the construction of this buttress wall was in progress, work was begun on a drainage tunnel to run into the bank west of the reservoir, at the berm level, for the purpose of draining away any water which might reach the reservoir from that direction. The construction of the buttress wall and the drainage tunnel at the berm level, occupied about one month, and occasioned some delay, but the reservoir was completed and ready for use early in December.

No further movement of the bank was observed until the very day, December 14th, 1894, the reservoir was being filled, when two cracks in the bottom were discovered near the south end of the buttress completed but a short time before, but the full significance of these new cracks was not discovered until after the reservoir had been filled.



CONCRETE BUTTRESS AT RESERVOIR NO. 3.
FIG. 2

Directions were given at once to have the reservoir emptied, which was accomplished on December 20th.

From the examination which followed the draining of the reservoir it was concluded that the difficulty was caused by the presence of water in the adjacent bank west of the reservoir basin, and steps were at once taken to construct a drainage tunnel behind the reservoir lining, as had been done at Reservoir No. 4 some months before, but this time on a grade parallel with the bottom of the reservoir basin.

This tunnel had its outlet connecting with the sewer at a point between the power-house and Dam No. 3, and was located along the margin of the bed of loose rock and clay which explorations had shown to exist in the bank on the west of the reservoir. The construction of this tunnel occupied several months, and when it had

FIG. 1.—RESERVOIR NO. 4. CRACKS IN PARAPET WALL AND WEST SLOPE. PHOTOGRAPH TAKEN SEPTEMBER 28TH, 1897.

FIG. 2.—RESERVOIR NO. 4. CRACK IN OUTER EDGE OF INCLINED ROADWAY, AND IN FACE OF SLOPE ABOVE SUB-RETAINING WALL, PHOTOGRAPH TAKEN SEPTEMBER 28TH, 1897.

been completed along the entire western margin of the reservoir to its northern extremity, at no point more than 100 ft. from the foot of the reservoir slope, it was thought that a satisfactory solution of the whole difficulty had been reached. Drain pipes were therefore placed in the bottom of the tunnel, and the excavated material was replaced, in the expectation that all needed drainage and protection work had been done.

While this tunnel work was in progress, surveys were begun for the purpose of determining the extent of the movement at both reservoirs. The indications of pressure against the reservoir walls were more pronounced at Reservoir No. 4 than elsewhere, but even there they were not so serious as to prevent the basin being at least partially filled from December, 1894, until the following September, The first evidence of pressure against the walls of the completed reservoir was noticed about the middle of January, 1895, or a few weeks after the basin was first filled, when a small crack appeared in the west parapet, near the old cable railway crossing. Other cracks soon appeared, increasing in number and size, until the middle of April, when the parapet was broken in several places and the lining was cracked, parallel with and about 6 ft. below the berm walk, for a distance of about 300 ft. An examination made at this time showed that the upper part of the lining and a portion of the parapet wall had been lifted clear of the ground and the wall tilted to the west.

The view shown in Fig. 1, Plate XX, was taken on September 28th, 1897, looking west from the north end of the dam at Reservoir No. 4, and shows the cracks in the parapet wall and the west slope, indicating a movement of the bank. The position of the concrete retaining wall, built under the slope lining during the construction of the reservoir, is indicated by the upper line of horizontal cracks about midway on the slope. The retaining wall for the roadway and the rip-rap face of the original excavation are shown above the parapet wall. Breaks in the rip-rap, due to the slide, can be traced at the upper left margin.

The photograph shown in Fig. 2, Plate XX, was taken on September 28th, 1897, looking south along the west slope of Reservoir No. 4, from a point on the parapet wall in front of the power-house. It shows the crack in the outer edge of the inclined roadway and in the face of the slope above the sub-retaining wall, and also the breaks in the parapet wall with the concrete slope lining crowded

nearly to the top of the wall by reason of the pressure of the moving ground. The maximum observed movement of the parapet wall from December 31st, 1894, to October 11th, 1897, was 3.24 ft.

Soon afterward, that portion of the lining above the crack was removed, and after this had been done the parapet returned nearly to its original position. In doing this work, a break was found in a 4-in. pipe, which had been laid under the berm walk for the supply of a series of jets around the margin of the basin; and, at another point on the same pipe, a branch was found to have been plugged with concrete instead of iron, and it was thought that these defects, by allowing the escape of water, had been instrumental in causing a settlement under the concrete lining and parapet. It was also recalled that the breaks appeared first near the point where a short section of the parapet had been built upon made ground, on account of a small slide which occurred before the reservoir was completed. These facts are mentioned simply to show that all were groping in the dark in search of some explanation of what was taking place before their eyes. The surveys made at this time indicated that a slight movement of the parapet wall had taken place, but it was not considered sufficient to explain the cracks in the concrete lining and parapet.

During June and July, 1895, the slight movement which had been noticed ceased so nearly that it was deemed safe to begin the repair of the reservoirs in order that they might be used. Accordingly, August and September were devoted principally to this work. While repairs were being made at Reservoir No. 3, additional drains were laid under the floor and on the west slope, as it was thought that they would prove a safeguard to protect the concrete work. This was a vain hope, however, for the repairs on the reservoirs had hardly been completed, and the basins partially filled with water, before it became apparent that the pressure from the adjacent banks was as great as ever. This was shown by the appearance of new cracks in the lining and parapet walls, and by the increased movement, as indicated by a re-survey of the range lines established during the previous January.

The repairs referred to above included an increased thickness of concrete upon the reservoir floor, making a total thickness of about 10 ins. Prior to this time the foundation of the buttress moved with the pressure. The subsequent movement caused the wall to break at the angle, from 4 to 6 ft. above the reservoir floor. See Figs. 1 and 2, Plate XXI.

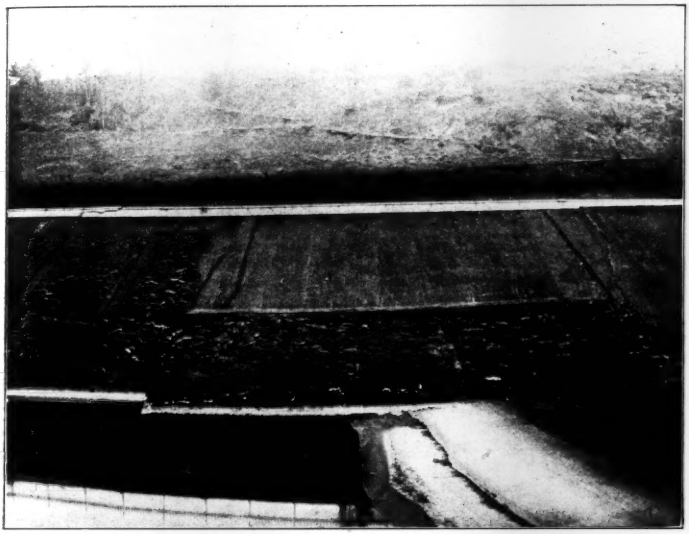


FIG. 1.—RESERVOIR NO. 3. HORIZONTAL CRACK IN BUTTRESS, AND BREAKS IN WEST SLOPE AND PARAPET. PHOTOGRAPH TAKEN SEPTEMBER 28TH, 1897.

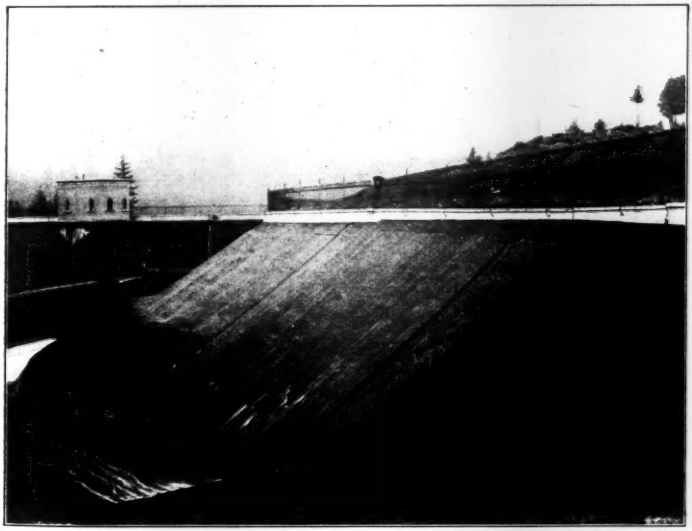


FIG. 2.—RESERVOIR NO. 3. CRACKS IN BUTTRESS, AND IN PARAPET. PHOTOGRAPH TAKEN SEPTEMBER 28TH, 1897.

This view was taken on September 28th, 1897, looking west from a point on the east parapet of Reservoir No. 3, opposite the buttress. It shows the horizontal crack in the buttress and the principal breaks in the west slope and parapet. The photograph shown in Fig. 2, Plate XXI, was taken on September 28th, 1897, looking south from a point on the parapet at the north end of Reservoir No. 3. The cracks in the buttress are shown on the left. At the right margin can be seen the northern limit of the broken parapet and slope lining. The maximum observed movement of the parapet from December 31st, 1894, to October 11th, 1897, was 1.69 ft.

From statements already made, it will be seen that, from the first, numerous theories were advanced to explain the cause of the difficulty with which the engineers were contending. At one time, the movement at Reservoir No. 4 was thought to be due entirely to a deposit of "quicksandy clay" behind the buttress which had been built under the lining before the reservoir was completed. The tunnel was then draining considerable water from the vicinity of this deposit, but apparently without effect.

By direction of the chief engineer, during March, 1895, the writer corresponded with Robert L. Harris, M. Am. Soc. C. E., since deceased, inquiring if his process for handling quicksand could be used to advantage in solidifying the material in situations similar to this, which was described to him. Mr. Harris stated in reply that he did not think that his process would be applicable, and said he thought the hillside was moving forward on a seam in the clay and that the movement was caused by a "hidden spring," as occurred in a case in his own practice where he ran a tunnel to the spring, and, by removing the water, stopped the slide.

The cause of the movement suggested by Mr. Harris was not deemed to be within the bounds of possibility, for the thought of the movement of the whole mountain side was an alarming one, but when, during the autumn, it was noticed that the movement was still in progress, even after the lapse of several months of dry weather, a new and more vigorous search was begun, in order to determine the point where the slide could have originated.

The writer was cognizant of the statements that had been made, to the effect that the whole hillside was part of an old slide, but not for a long time, and only after repeated examination, was he able to deter-

mine, even approximately, the boundaries of the ground which might be said to form part of an "old slide." The ground adjacent to the reservoirs was examined carefully for surface cracks which it was thought would certainly appear at some point within reach of an ordinary slope upward from the reservoir bottom. That such surface signs should not be found within a distance of 600 ft., or that the slope of the break should be flatter than 1 on 3, was then undreamed of and not deemed to be within the bounds of possibility.

Gradually the limits of the examination were enlarged, until, on September 23d, while exploring the vicinity of the old cable track on Kingston Avenue, the writer detected a slight bend in the rails.

During the week following this discovery further explorations were made, and instrumental surveys as well, which resulted in locating the head of the slide in a marshy depression in the hills about 600 ft. west of the cable track, and some 1 700 ft. or more from the reservoirs. The "round top" or knoll apparently at one time formed a part of the ridge. The marshy depression in front of the knoll, to which reference has been made, is semi-circular in shape, about 300 ft. long and from 30 to 60 ft. wide. The rim of the basin at the southeastern end was not more than 2 or 3 ft. above the level of the marsh, and therefore the water stored could not have been of a greater depth. When discovered, there was no water standing on the surface, but the ground was moist in places and covered with swamp grass, weeds and brush. Subsequently, the peaty formation was found to be from 15 to 20 ft. deep, with clay underneath.

That the movement must have been in progress for months, or years, was shown by the deflection in the alignment of the cable track at the center of the slide, which amounted to 2.2 ft. maximum, as determined by the instrumental observations, assuming that the track was laid out originally as a tangent for the entire distance along Kingston Avenue, which is known to have been the case. The examinations also showed the entire absence of all surface cracks, parallel with the reservoirs, at any point between the reservoir basins and the swampy ground at the head of the slide. A sign of the movement was found at one intermediate point, however, as shown in Fig. 2, Plate XIX. This point was at an angle in the cable railway in its western course up the hill, and about 500 ft. from Reservoir No. 4. The angle in the track at this point is about 18°; the distortion of the rails, however, when first

noticed, was about one-third as great as shown in the photograph, the latter not having been taken until June 24th, 1898, or nearly three years after the movement was first discovered.

As the result of these examinations, it was decided to begin at once a series of instrumental measurements to determine whether the sliding ground was really as extensive as recent developments indicated. In pursuance of the plan then adopted, several range lines were established, with their terminal points widely removed from the vicinity of the suspected ground, each line having a number of intermediate stations located so as to be observed easily and accurately, and at the same time be on supposedly moving ground.

During the latter part of 1895, and the first six months of 1896, twenty-nine of these range lines were established, about half of which were observed at intervals of about one month for a period of from three to five years, and several have been observed to the present time. The lines, at first, were established so as to cover a wide range of territory, for it was thought possible that the movement of the hillside might be more extensive than then appeared from any visible signs, and therefore several of the ranges were extended to the summits of the highest ridges in sight upon the north and south sides of the supposed sliding ground. After a few months' trial the observation of these long-range lines was discontinued, for it soon became evident that the movement was confined to the ground lying to the west of the reservoirs and within the limits of the ravines extending west from the City Park.

Some range lines were also established early in 1895, with reference to the movement of the parapet wall. When it became apparent that the movement extended for a considerable distance from the parapet walls, preference was given to observations on lines having points set in the ground, for it was noticed that the pressure against the parapet wall was not uniform, and that, in places, the wall was being tilted to the west.

In 1895, when most of these range lines were established, no surface signs of movement, other than the curvature in the cable-railway tracks and the breaks in the lining and parapet walls of the reservoirs, were apparent at any point, excepting a few small cracks in the ground in the vicinity of the marsh at the head of the slide. The surface of the ground, between the reservoirs and the cable road on

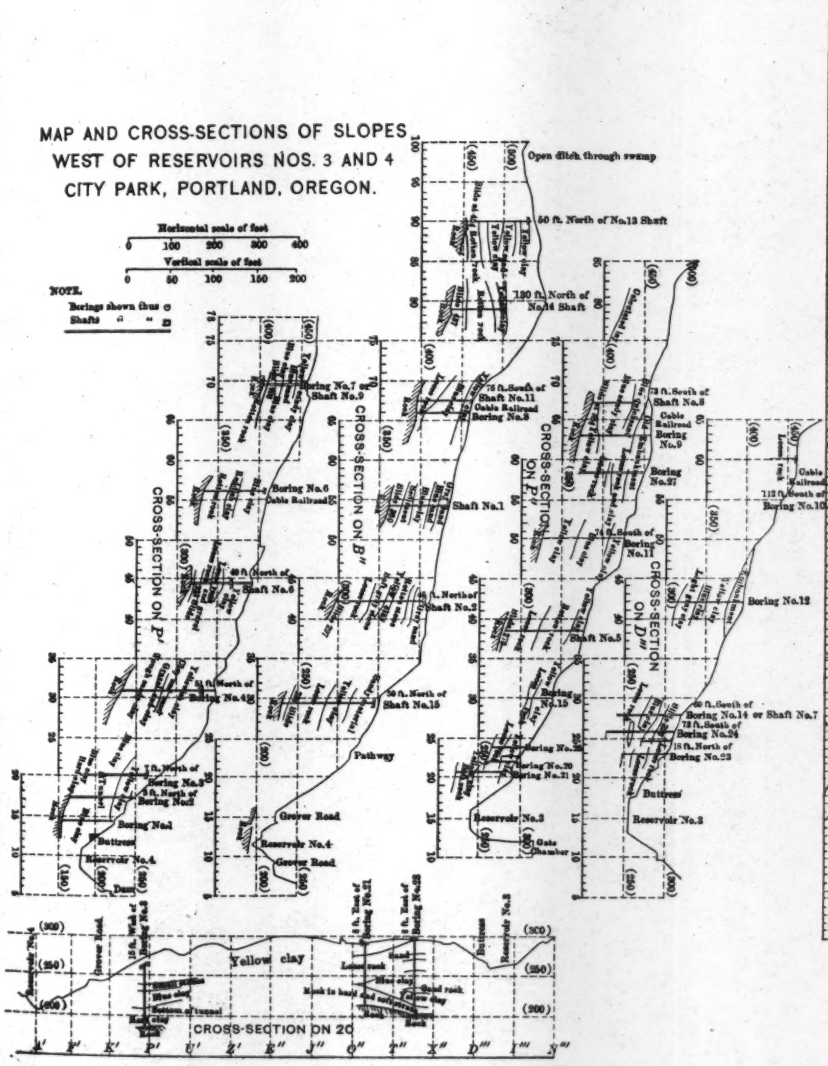
Kingston Avenue, was searched carefully for cracks which might indicate the extent of the movement, but, contrary to all expectations, no crevice could be detected along the high ground adjacent to the reservoirs and parallel with them, and, strange as it may seem, no such crack has ever been observed, with the exception of one, about 125 ft. long, near the site of the old elk barn and about 200 ft. west of the power-house and Dam No. 3. This crack did not appear until May, 1896, or eighteen months after the reservoirs were excavated, and could not be seen long, for, having been only a trace along the surface, it was obliterated entirely in a few months. Other cracks, of a larger size, were found, about this time, around the margin of the depression, or marshy ground, at the head of the slide, and these increased to such an extent that during 1897 the outline of the movement in that vicinity could be traced by an almost continuous break in the ground.

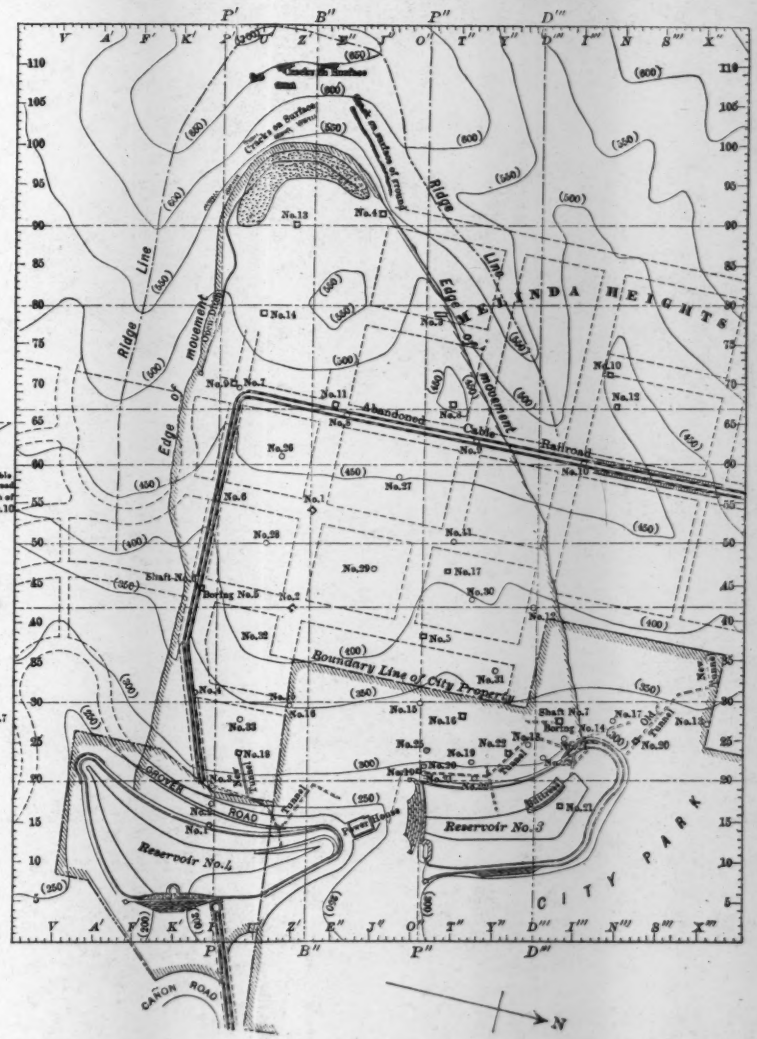
In the autumn of 1895, or as soon as it became apparent that the movement extended to such a great distance from the reservoirs, the chief engineer directed that arrangements be made to determine the depth of the slide, as well as the area, in order to ascertain whether or not the movement was along or near the surface of bed-rock.

The plan decided upon for securing this information was to drill through the overlying material, and into the rock, with an ordinary well-boring machine, a water-jet being used to remove the earth and pulverized rock. This work was in progress from September 27th, 1895, until March 31st, 1896. In all, twenty-five holes were drilled, aggregating 1 710 lin. ft. (See Plates XXII and XXIII.) These borings were located along the cable track, and eastward, and, subsequently, two of them were found to be outside of the limits of the moving ground. The contract for this work specified that the holes should be drilled through the overlying strata of clay and loose rock, and 5 ft. into the solid bed-rock, so as to make it certain that the drill did not stop at each small boulder it encountered. The holes were 4 ins. in diameter, and were cased with wrought-iron pipe which was driven until the lower end was in close contact with what was thought to be bed-rock.

The prices paid for this work were $1.25 per linear foot for earth or loose material, and $3.00 per linear foot for solid rock, where the holes were drilled deeper than the 5 ft. specified, as was done in some instances. These prices included the use of the plant, and labor only,

MAP AND CROSS-SECTIONS OF SLOPES WEST OF RESERVOIRS NOS. 3 AND 4 CITY PARK, PORTLAND, OREGON.





all other expenses, for casing, and for wood and water for the use of the engine, being extra, the total cost averaging $1.68 per linear foot.

After the holes had been drilled and cased, the next thing was to devise some plan by which the depth at which the movement was taking place could be determined with accuracy, and the following was decided upon: To measure the depth of the holes, a small pipe, 1 in. in diameter, was fitted in 10-ft. sections for ease in handling. At each end of one of these 10-ft. sections there was fitted a flange, about 3¾ ins. in diameter, which nearly filled the inside of the well casing. To this flanged section of pipe the other sections were coupled for a handle, and with this handle the flanged section was passed down the well casing to bed-rock.

A trial of this sounding rod was made soon after each boring was completed, and in all cases it would pass freely to the bottom of the hole. Subsequently, this trial was repeated at intervals of about one month, but, after one or two trials, it was found that the pipe flanges would stick in the casing and prevent the rod from reaching the same depth as upon the first trial. This was understood to mean that a bending of the well casing, caused by the movement of the slide, was taking place at about the depth reached by the flanged rod, the flanges on the bottom section preventing the rod from passing the bend in the pipe.

Subsequent explorations showed conclusively that measurements of the depth of the slide made with these rods were practically correct. In three instances where bends in the casing occurred the pipes were dug out afterward and were found to be badly bent or broken within 1 ft. of the depth determined by the sounding rod. In most of the borings made the movement was found to be taking place at or near the surface of the bed-rock, and at depths varying between 50 and 112 ft. below the surface of the ground. In some of the borings, water would rise in the casings after they were completed, indicating the presence of water pockets at various places in the sliding ground.

The necessity for an improvement of the drainage of the sliding land district was early recognized, and, before the beginning of the autumnal rains in 1895, by permission of the property owners, a drain was dug through the southern rim of the basin or marshy ground at the head of the slide, so as to convey away any surface water which might otherwise accumulate there during the winter. That water had

accumulated there formerly has been testified to by those familiar with the locality, but probably the depth did not exceed 2 ft. Hand-auger borings were also made in the marsh, and showed that the peaty formation on the surface did not exceed a depth of 20 ft. and was underlaid by a bed of clay.

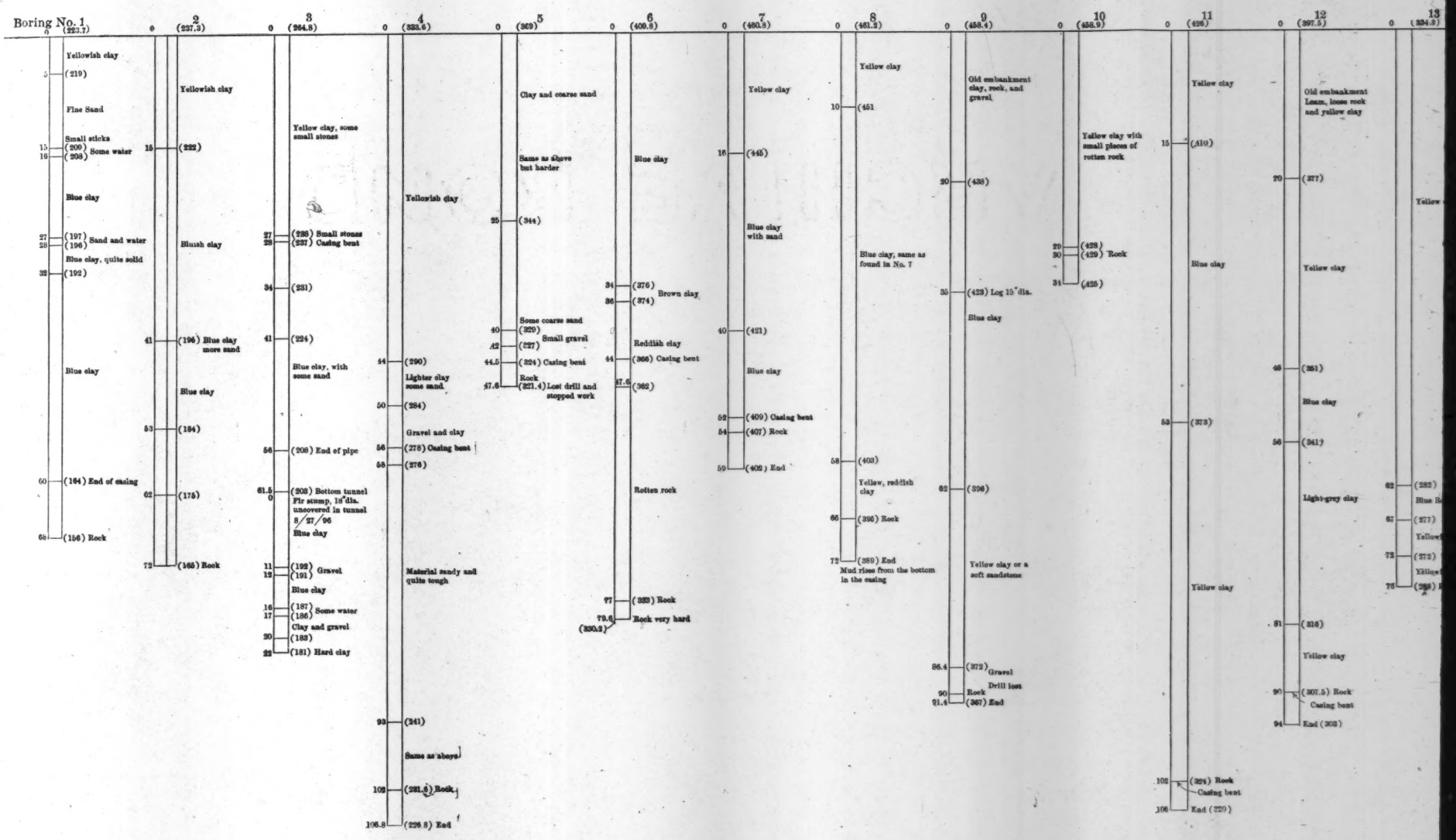
While this work was in progress the King Real Estate Association was besought for permission to fill the pool in the depression left by them west of the cable railway embankment at the time the grading was done and the culvert built. This culvert or sewer was not laid deep enough to drain the bottom of the ravine, and, in consequence, a pool of water, 30 to 40 ft. in diameter, and from 3 to 5 ft. deep at the center, had accumulated there. This pool was kept full constantly by the drainage from several small springs which came to the surface in the bottom of the ravine above the railway crossing. The request of the chief engineer for permission to fill this pool, at the expense of the Water Committee, was granted by the property owners, after some little time had been taken to consider the matter, and the work was begun at once. Drains for the removal of surface water at points adjoining the reservoirs on the west were also built and connected with the sewers constructed during the previous year.

During the winter of 1895–96, and, in fact, during the entire year of 1896, the only construction work done in connection with the sliding land was to see that all ditches and drains were kept open so as to carry away all surface drainage. Surveys of the range lines were also made regularly each month to determine the rate of the movement. As one result of these repeated observations, it was noted that there was a marked increase during the winter or rainy months—say, November to May, inclusive—and a corresponding diminution of the movement during the remaining months of the year, when the rainfall was considerably less.

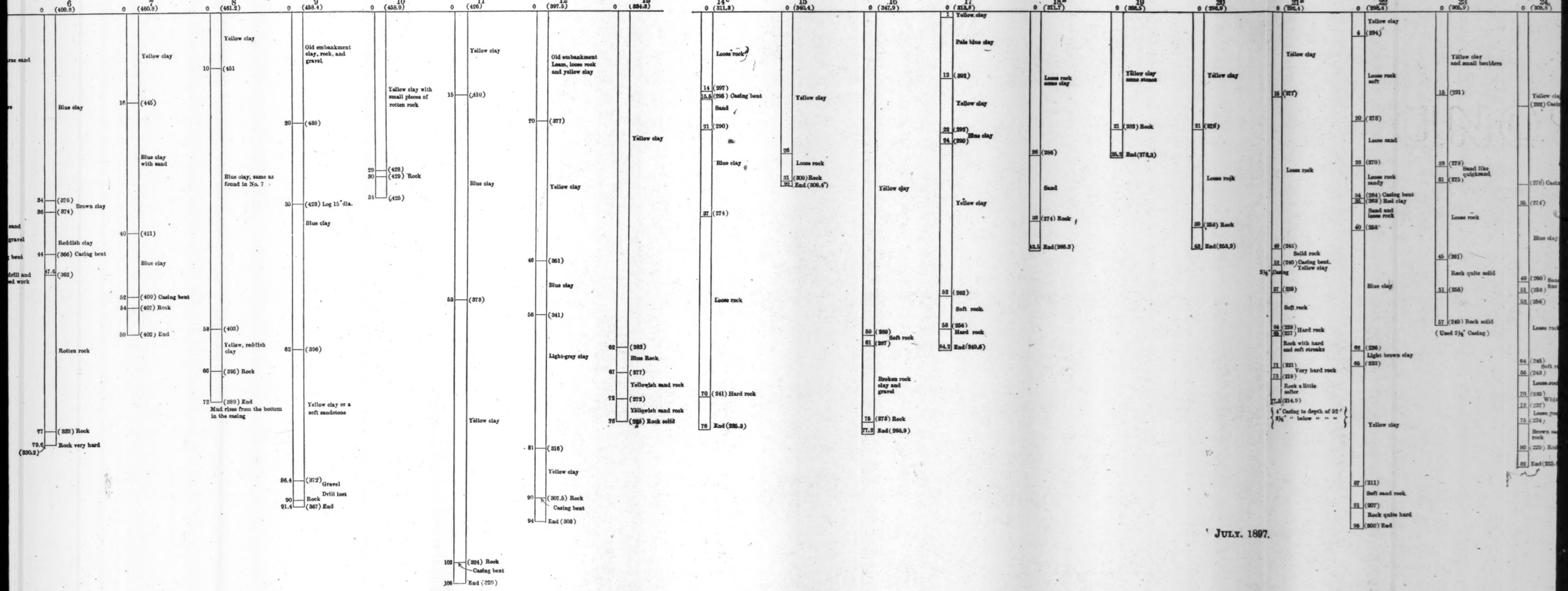
During 1896, the cracks in the reservoir linings were observed to be increasing slowly in number and size, and, for the purpose of determining what effect, if any, would be produced thereby, the water in the reservoirs was drawn down and at times the basins were entirely emptied. Owing to the cracked and broken concrete lining, and the absence of water in the basins, the reservoirs soon began to present an unsightly appearance.

It may be well to explain here that the quantity and quality of the

BORINGS WEST OF RESERVOIRS

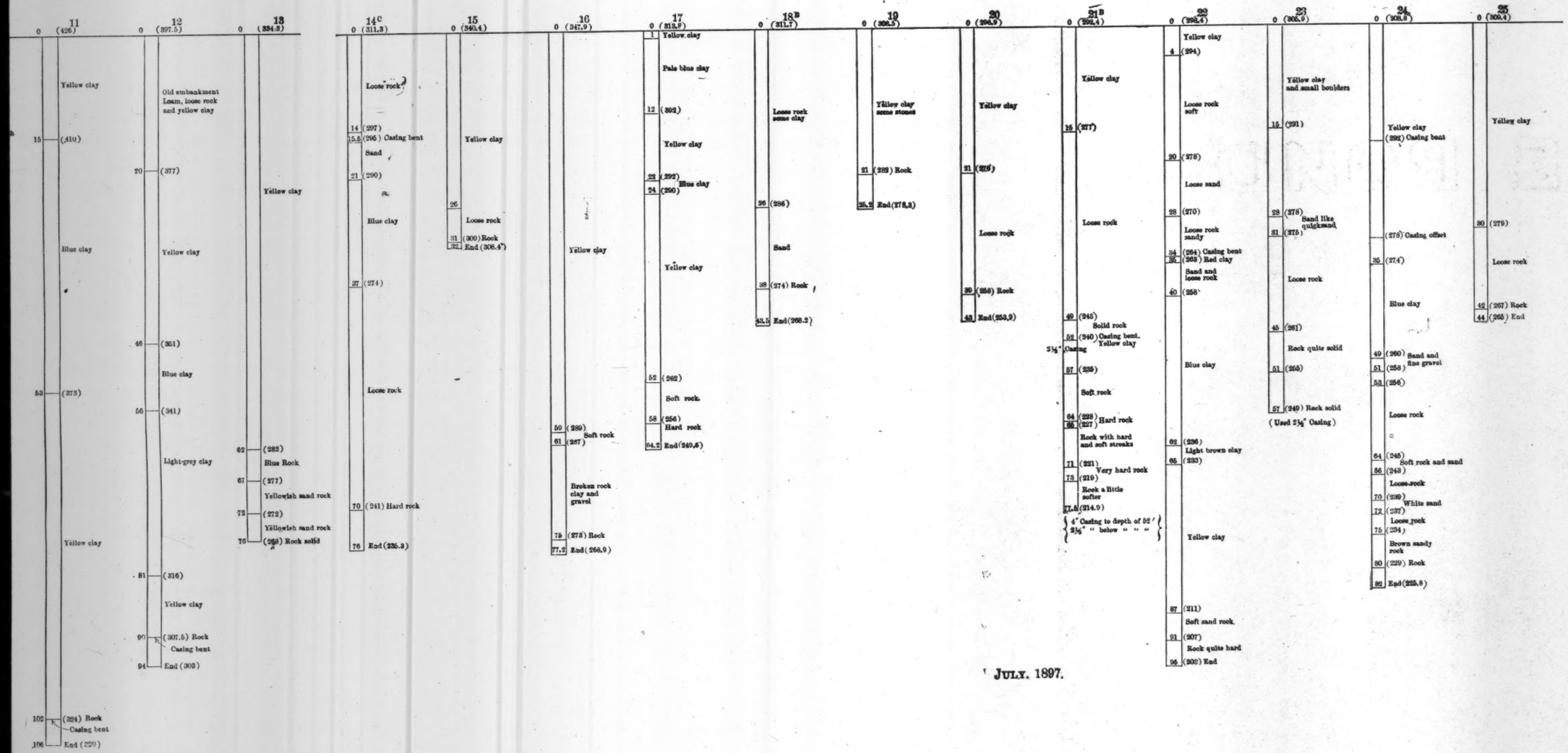


BORINGS WEST OF RESERVOIRS NOS 3 AND 4 MADE IN 1895 AND 1896.[1]



[1] JULY, 1897.

NGS WEST OF RESERVOIRS NOS 3 AND 4 MADE IN 1895 AND 1896.[1]



[1] JULY, 1897.

water delivered to the city has never been affected by the defective condition of the City Park reservoirs. The reservoir system, as designed and built, provides that the reception of water from the main conduit, and its distribution to the different parts of the city, should all be done through the medium of wrought-iron or steel tanks, one of which is located in the interior of the gate-house connected with each reservoir, and in which all mains are centered. The reservoir basins were intended for storage purposes only. In some instances the inflow and outflow are through the same pipe, hence a circulation of water through the reservoir cannot be maintained without overflow into the sewers. The total estimated capacity of the four reservoirs connected with the system is 66 000 000 galls., or four days' supply for the city.

The temperature of the water delivered in the city during the summer months is also appreciably cooler* when distributed through the gate-chambers direct than it is when stored in the reservoirs, even for a few hours, and hence is more satisfactory. From the foregoing statements, it will be seen that the delay in completing the reservoirs for use has in no way detracted from the general excellence of the water supply furnished to the city during the past nine years.

Early in the spring of 1897 the proof that water in the underground springs, fed by percolation from the surface, was a prime factor in producing the slide, if not the only originating cause, became so conclusive that, on April 27th, the writer addressed a letter to the chairman of the Water Committee, explaining at considerable length the discoveries thus far made, and the reasons for believing that the removal of the water by a thorough system of drainage would stop the slide. In this letter, reference was made to a drainage project and estimate of cost which had been prepared by the writer during the preceding year.

A few weeks later, action was taken by the Water Committee, looking to the engagement of a consulting engineer to examine the sliding ground and report on the best means for the cure of existing conditions, which happily resulted in securing the services of G. H. Mendell, M. Am. Soc. C. E., Colonel, Corps of Engineers, U. S. A. (since deceased), for the investigation of the problem.

Colonel Mendell examined the reservoirs first in July, 1897, and also at intervals thereafter, and at his direction certain additional

* The temperature is from 55 to 62° Fahr.

surveys and explorations were undertaken which resulted in the discovery of a large number of interesting data regarding underground conditions, and the depth and cause of the slide.

In order to avoid all uncertainties regarding the character of the bed-rock and the overlying earth, Colonel Mendell approved of the suggestion that wells or shafts be excavated to bed-rock by hand, and be made $3\frac{1}{2}$ x $3\frac{1}{2}$ ft. in size, instead of the 4-in. borings used formerly. This method of procedure proved to be very successful, and, in the main, was followed in all subsequent explorations, although at one time the boring machine was again used for a few weeks in order to expedite the work.

The excavation of the open shafts, although more expensive than the smaller drilled holes, was in every way more satisfactory, for it was then possible to determine the character of the bed-rock and the overlying material more satisfactorily than could have been done in any other way. Especially was this true of the seam of blue clay adjoining the bed-rock, forming the bed of the "ancient slide," which has played such an important part in all later studies of the problem.

Work on these wells or shafts was begun on July 19th, 1897, and prosecuted almost continuously from that time until January 24th, 1899, a part of the time with two crews of men. An ordinary winch and bucket, worked by hand, was used for removing the excavated material. It was found necessary to use curbing, from the surface down, 2 x 8-in. fir plank, notched at the corners, being used for this purpose. Twenty-two shafts, in all, were excavated, having an aggregate depth of 1 497 ft., with 454 lin. ft. of tunnel connected with the same. While this work was in progress, nine additional borings, aggregating 677 lin. ft., were made for the purpose of expediting the work and in order to fill in certain areas where the character of the material was not known with sufficient accuracy and it was mainly desired to determine the depth of the earth overlying the bed-rock. The location of the shafts is shown on Plate XXII, and in Fig. 3, which shows some profiles through the center of the slide. Sections of the shafts and borings are shown on Plate XXIV.

The shaft excavations and borings were made experimentally, the discoveries at one shaft awakening inquiry, and suggesting investigations at another point, and so on, until a much larger field was covered

SKETCH AND PROFILES OF MOVING GROUND
WEST OF RESERVOIRS NOS. 3 AND 4 CITY PARK, PORTLAND, ORE.
SHOWING GENERAL CHARACTERISTICS OF THE MATERIAL BETWEEN THE SURFACE AND BED-ROCK,
AS DETERMINED BY BORINGS AND SHAFT EXCAVATIONS COMPLETED IN 1898.

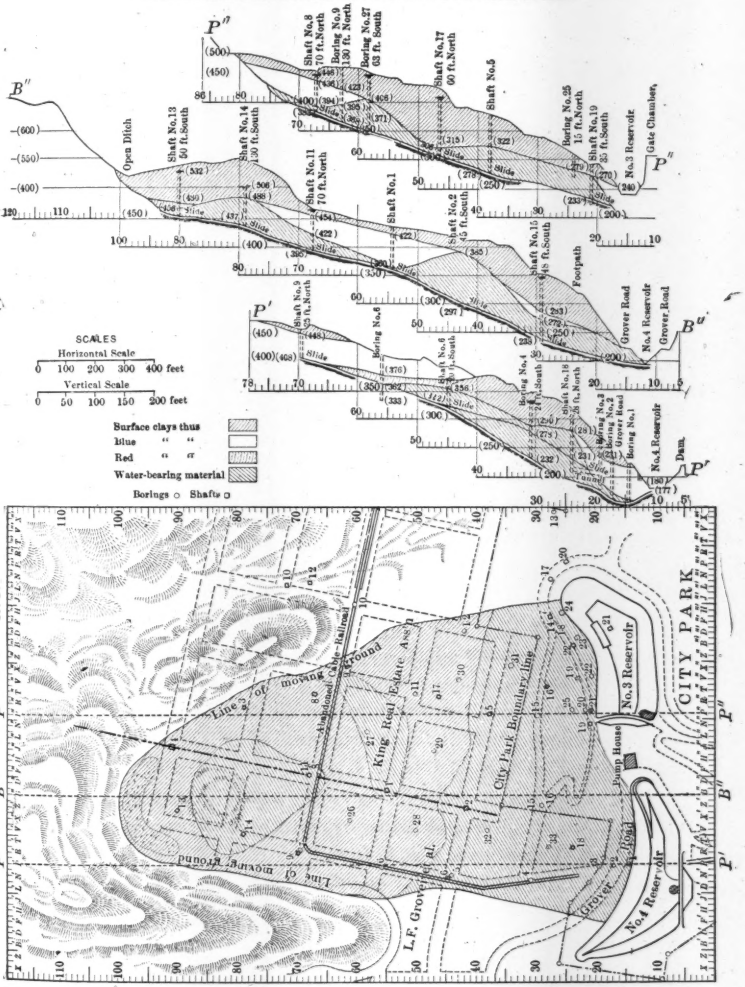


FIG. 3.

and more work done than was anticipated at first. A number of surprising developments resulted from this series of investigations, particularly as to the character of the material between the surface and bed-rock, and the existence of a well-defined seam of clay near bed-rock which gave ample evidence of being the bed of the slide. In certain localities an unexpectedly large quantity of water was found in connection with the clay seam lying upon or near the surface of the bed-rock, and much attention was given to investigations of the source and extent of this underground water supply, for by this time it had come to be fully realized that water in the seams of the material composing the hillside was, chiefly, the originating cause of the movement.

The first excavation made, Shaft No. 1, was on the line between the King and Carter claims, near the axis of the slide and about 900 ft. west from Reservoir No. 4, while Shaft No. 2 was about 250 ft. farther east. These shafts were located at these points simply because they would thus practically cover a large part of the territory adjacent to the reservoirs which remained unexplored at the conclusion of the work of the previous season. The discoveries made at these points were most surprising, and had an important bearing upon all the later investigations. A detailed statement of the peculiar characteristics of the material found in each shaft and boring will be found on Plate XXIV, but it may be of interest to refer briefly to some of the more important discoveries made.

Shaft No. 1.—The excavation of Shaft No. 1 was begun on July 19th, 1897, and completed on January 31st, 1898. The elevation of the surface was 435 ft. above the city datum in use in 1894, which was about the elevation of low water in the Willamette River. From the surface to a depth of 13 ft., a grayish quicksandy clay was found. It should be stated that the original surface of the ground had been removed when the streets and lots were graded and the consequent cutting down of ridges and filling of ravines was accomplished. The total depth of excavation was 75 ft.

At 13 ft. a hard, blue, sandy clay was found, upon the top of which a small quantity of water collected. At 25 ft. the blue clay was found mixed with fragments of rock, quite soft, and nearly disintegrated. At 26 ft. a crack, ½ in. wide, was noticed, having a northeast and southwest course, but it was soon lost. The ground at that depth was quite

BORINGS MADE AND SHAFTS SUNK WEST OF RESERVOIR

BORINGS

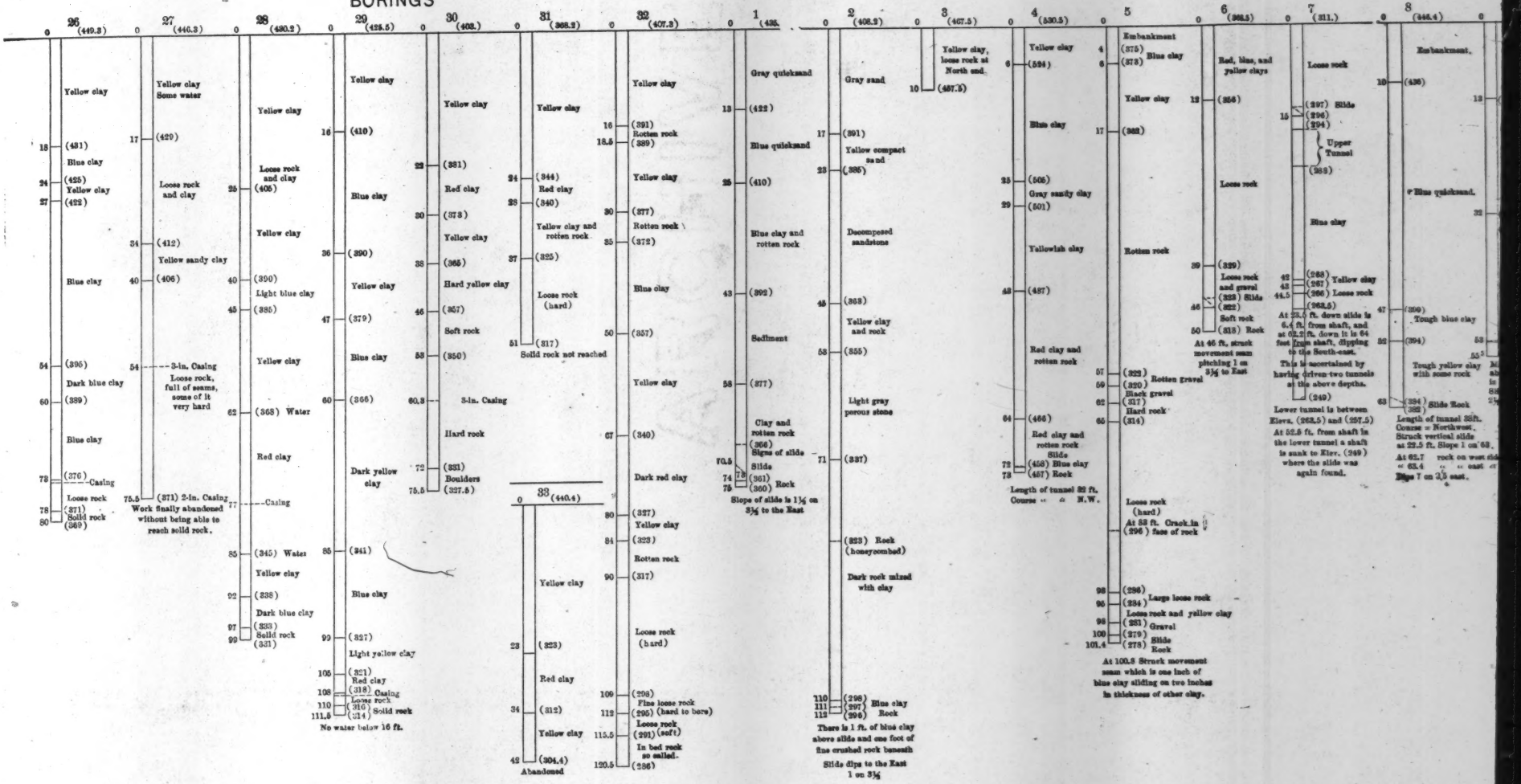
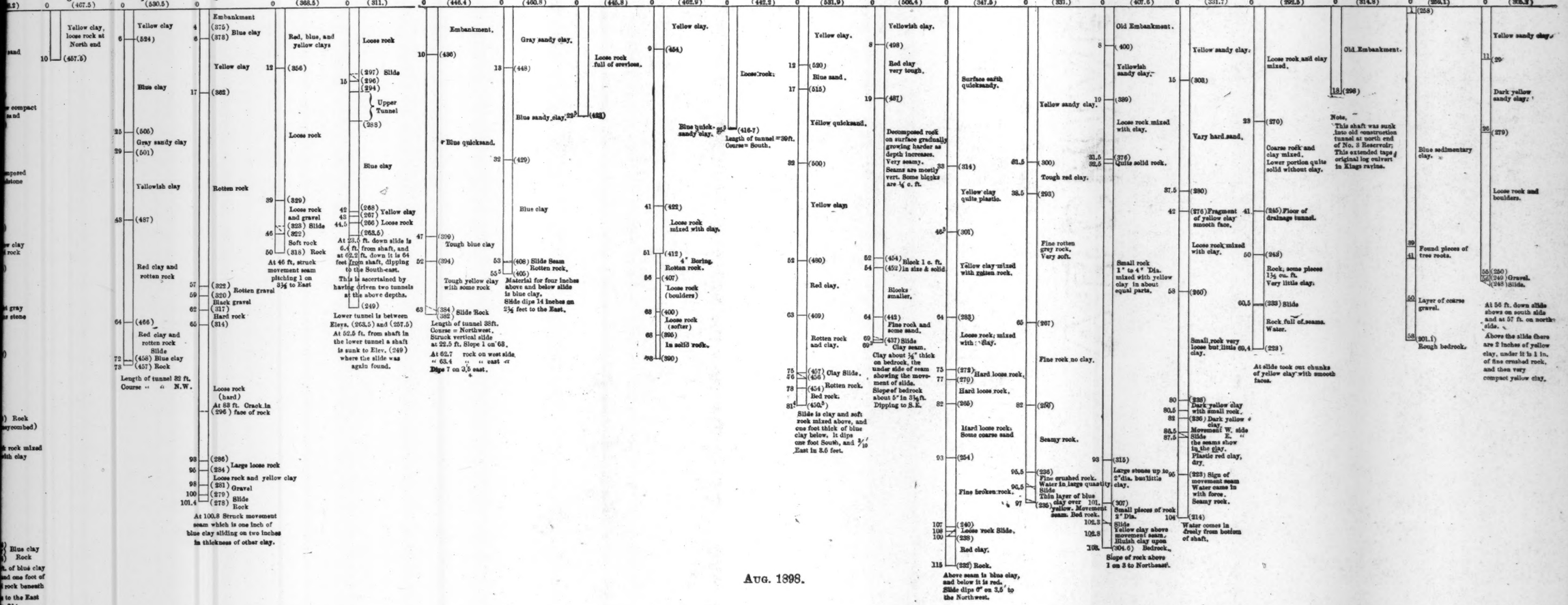


PLATE XXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIII, No. 984.
CLARKE ON A PHENOMENAL LAND SLIDE.

ORINGS MADE AND SHAFTS SUNK WEST OF RESERVOIRS NOS. 3 AND 4 IN 1897 AND 1898.

SHAFTS



AUG. 1898.

solid. The blue clay, mixed with rock, continued to a depth of 43 ft., where was found a blue, sedimentary clay without any admixture of broken rock. At this depth some small fragments of wood were found, which were apparently of fir. They were much discolored, but well preserved, the grain of the wood showing distinctly. The pieces of wood found in the shaft aggregated about ½ cu. ft. in volume. This blue, sedimentary clay continued, practically unchanged, to a farther depth of 15 ft., or to 58 ft. below the surface. At 50 ft. foul air was encountered, making it necessary to put in a ventilating shaft, which consisted of a small box in one corner of the shaft, at the lower end of which a small fire was maintained. A few feet below this level a thin layer of material, very black, tough and dry, was found. At 58 ft. was found a mixture of blue and yellow clay, mingled with broken rock in which a water-worn or well rounded stone, 2½ ins. in diameter, was found. Until this stratum of clay and rock had been penetrated 10 ft., or to a depth of 68 ft. below the surface, little difficulty was caused by ground-water, which collected in the shaft only at the rate of from 25 to 40 galls. per day. At 68 ft., the clay at the west side of the shaft was dark blue, unmixed with rock, some pieces of clay having smooth faces. At this depth the flow of water increased suddenly, so that the shaft was filled to a depth of 18 ft. in one night. The next day the water was bailed out, and excavation was continued. During the following night the shaft was filled again, this time to a depth of 20 ft., the flow being at the rate of 1.8 galls. per minute. After contending with this flow of water for several hours, the shaft was deepened an additional foot, or to 69 ft. below the surface, more pieces of clay being found with smooth upper faces. At 6 P. M. on August 6th, 1897, just as the men were quitting work for the night, the flow of water increased suddenly to 40 galls. per minute, rising in the shaft 33 ft. in one hour. The next morning it stood within 8½ ft. of the surface of the ground, making a total rise of 60½ ft. in 12 hours.

The flow of water in the shaft was so strong as to render hopeless the task of lowering it with an ordinary windlass and bucket, and, therefore, it was decided to install a deep-well pump, run by steam power, in order to drain the shaft sufficiently to proceed with the excavation. This was done at once, and the pumps were in operation on August 19th, but at that time there was but little appreciation of

the magnitude of the task, for it was not until January 31st that the work was completed and the pumps removed.

Beginning with two small pumps, with about 3½ x 18-in. cylinders, the number of pumps was increased to four before any appreciable headway could be made. The estimated capacity of all four pumps was 58 galls. per minute. By using four pumps it was practicable to lower the water nearly to the bottom of the shaft without continuous pumping, but, by resting half an hour, the water would rise in the shaft, and pumping had to be resumed. By noting the diminishing height to which the water rose during the half-hour interval of rest, some idea was obtained as to the progress which was being made in draining the underground reservoir. At first, the water would rise from 14 to 15 ft. in half an hour, while toward the end the rise was not more than one-quarter of that amount in the same time. During a portion of the time, in order to expedite the work, the pumps were run day and night.

Toward the latter part of January, 1898, the flow diminished to such an extent that it was practicable to dispense with two of the pumps, thus giving room to work in the bottom of the shaft while pumping was in progress. After an effort lasting several days, the layer of loose rock, underlying the clay seam heretofore mentioned, was penetrated, and, at a depth of 75 ft., bed-rock was reached and found to be comparatively smooth and free from fissures. Above the bed-rock, at the east side of the shaft, a plainly defined movement seam was uncovered, having 5 ins. of dark blue clay above the line of cleavage, with fine broken rock between the clay and the solid bed-rock. The inclination of the clay seam was eastward, approximately 2.5 ft. in the width of the shaft.

The line of cleavage in the stratum of clay was defined very clearly. The upper and lower portions of the stratum could be easily separated along the line of cleavage, and the faces which had been in contact presented uniformly a smooth and glazed appearance. When first exposed, the surfaces were bright and shining, as though they had been varnished, but when dry the color was of a dull and leaden hue. There were slight ridges or inequalities in the surfaces, running east and west, showing the presence of grains of rock or some hard substance, and indicating also that movement had been taking place along the plane of the two surfaces. The excavation was

carried below the main water-bearing stratum, which appeared to be above the clay seam.

On January 31st, 1898, after the excavation had been carried to bed-rock, the pumping was discontinued. The total quantity of water pumped from the shaft from August 19th, 1897, to January 31st, 1898, was, approximately, 3 925 000 galls.

Measurements made on February 16th, 1898, showed that the water level in the shaft had been lowered 51 ft. by the removal of nearly 4 000 000 galls. of water. After the cessation of pumping, the water continued to rise slowly until March 2d, 1899, when it stood at Elevation (435), or level with the surface of the ground. It was afterward noted that there was a slight falling away during the dry season, followed by a complete recovery during the winter.

Shaft No. 2.—While the work on Shaft No. 1 was in progress, the other shafts were also being excavated. Shaft No. 2, one block east of Shaft No. 1, and also on the line between the King and Grover claims, the surface elevation being (408), was begun on July 19th, and completed on August 20th, 1897. The total depth of the shaft was 112 ft.

The same grayish, quicksandy material found near the surface of Shaft No. 1 was found here to a depth of 17 ft., and below that a stratum of yellowish sand, 6 ft. thick. At 12 ft. below the surface, a vertical crack, ½ in. wide, running northeast and southwest, was noticed, the material west of the crack being yellow, while that on the east side was of a bluish color. The excavation followed the crack for about 7 ft. only. From 23 to 45 ft. the material was principally very hard sand. From 45 to 53 ft. yellowish clay mixed with broken rock was found. From 53 to 71 ft. the material was principally sand, similar to that between 23 and 45 ft., and, in places, was very hard to pick. From 71 to 105 ft., grayish, honey-combed rock, full of seams, with some earth, was found. In some places the material was hard, and in other places it could be picked easily. At 105 ft. the material was quite loose, with some pieces of dark blue clay. Here 20 galls. of water collected in one hour. From 105 to 110 ft. small stones mixed with coarse sand were found. At 112 ft., solid rock, free from seams, was found, with a dip to the east of about 1 on 3.5. There was a stratum of dark blue clay 1 ft. thick on top of the rock. The line of the slide was defined clearly near the bottom of the

clay. Since the completion of this shaft the water has never collected to a greater depth than 7.5 ft.

Shaft No. 3.—In excavating Shaft No. 1 it was noticed that the water at the bottom of the shaft came from the northwest, which fact called renewed attention to the springs in that direction and west of the old "pool," to which reference has already been made. It was therefore decided to sink Shaft No. 3 near the west end of the ravine in which these springs were situated.

The elevation of the surface was (467.5). The depth of the excavation was 10 ft. Excavation was begun on August 9th, 1897, and abandoned the next day, as the location was found to be too near the margin of the moving ground to afford the information desired.

For the first 6 ft. yellowish clay and logs were found intermingled. A small quantity of water came from seams in the rock at the north end of the pit. When work was suspended, the materials found were clay at the south end of the pit and loose rock at the north end.

In regard to the springs, the owner of the property stated that the flow of water was much less than when he first examined the ground, perhaps forty years ago, the springs formerly furnishing a water supply for some of the buildings on the eastern border of the property.

Shaft No. 4.—The point next selected for examination was a few feet north of the claim line and a short distance from the western end of the moving ground, the surface elevation being (530.5). The depth of the excavation was 73 ft. Excavation was begun on August 11th and completed on August 23d, 1897.

From the surface down for a depth of 6 ft., the material was yellow clay, and below that was found 19 ft. of bluish clay. From 25 to 43 ft., were found gray and yellow clays which were quicksandy in places. From 43 to 72 ft., the material was chiefly red clay mixed with small pieces of porous rock. Between 44 and 46 ft. the clay was very plastic and was yellow when dry. At this level, 150 galls. of water collected in one night. From 72 to 73 ft., a layer of dark blue clay resting upon rock was found. The surface of the rock could be picked to the depth of 6 ins., but below this it was hard. The rock dipped slightly to the east. The line of the movement seam was defined clearly between the depths, 72 and 73 ft., and consisted of about 1 in. of red or yellow clay resting upon blue clay, with solid rock underneath.

This shaft being near the border of the moving ground, it was

next determined to run a tunnel along bed-rock until the northern limit of the slide should be reached, if this could be found within a reasonable distance. The slope of the rock being easterly, the tunnel was started in a northwesterly direction, in order to provide for drainage. This work was begun on August 26th and completed on September 15th, 1897. The securing of proper ventilation for the shaft and tunnel proved to be quite a hindrance. As the tunnel was expected to be only a temporary affair, it was made as small as could be worked conveniently, about 3 x 5 ft., and was not timbered thoroughly. The material encountered was similar to that in the shaft, being chiefly reddish clay mixed with fragments of porous rock. The tunnel followed the top of the stratum of blue clay, the dividing line between the blue clay and the red clay and gravel being plainly marked.

At 28 ft. from the shaft was found a seam of blue clay, 1 in. thick, with smooth faces, showing pressure, but not furrowed. The dip of the seam was 3 ins. in 2 ft., and to the northeast. At 32 ft. a plainly defined crack was found which was nearly vertical and crossed the tunnel nearly parallel with the break in the surface of the ground, which marked the margin of the slide opposite that point. When approaching this crack the breast of the tunnel fell in, showing the line of the movement seam. Beyond the crack, the material was cemented and much harder than anything found before, some large pieces of rock appearing in the face. From measurements made at the tunnel level, and also at the surface of the ground, the slope of the run of the slide at that point was determined as approximately 9¼ vertical to 1 horizontal.

During the excavation of the shaft and tunnel, the water draining from the ground caused little trouble until a point near the bottom was reached. The quantity which collected each night gradually increased from 8 galls., when at a depth of 25 ft., to 120 galls., when at the bottom, with the exception of one night, already noted, when, at a depth of 44 ft., the quantity collected was 150 galls. The drainage was principally from the north. After the completion of the tunnel, the water collected for some days at the rate of 20 to 25 galls. per hour, and in about two weeks it had filled the tunnel and shaft to the depth of 17.5 ft. In February, 1898, the water stood at Elevation (514.9) or at a depth of 56.4 ft., but the ordinary depth was from 25 to 40 ft.

During the period covered by the observations, the variations in the water level at this shaft were much more marked than at any other point observed. The rise which took place within a day or two after a heavy rainfall was especially noticeable. As a possible explanation of this sudden rise, it may be noticed that the shaft is at the foot of a steep bank where the coating of fine surface earth is comparatively thin and the underlying material probably more than usually porous. The tunnel, having a direct connection with the crack reaching to the surface, may also have had some effect. It may be noted, also, that at this point, and at all other shafts, the surface around the opening was protected against direct drainage into the well.

Shaft No. 5.—Work at this point, located about 400 ft. west of Dam No. 3, was begun on August 23d and completed on September 21st, 1897. The surface elevation was 379.6. The depth of the excavation was 101.4 ft. For the first 6 ft. the excavation was through an old embankment made when the property was being graded. Next was found 11 ft. of yellow surface earth and below that 40 ft. of grayish, porous rock, growing harder toward the bottom, where it was full of seams, and similar to that found in Shaft No. 2 at about the same elevation. Next below, a 2-ft. layer of small stones and coarse sand, cemented, was found, followed by 6 ft. of hard basalt in small pieces, which could be removed with a pick. Below that, for a further distance of 30 ft., rock of the same general character continued. The fragments of rock were mostly small (one layer, several feet in thickness, consisting of pieces of rock ½ in. to 2 ins. in diameter and so loose as to require the use of curbing), except near the bottom, where larger blocks were found, some of which had to be broken before they could be removed from the shaft. Below the larger pieces of rock came 3 ft. of yellow clay mixed with small fragments of rock, and then came another layer, of about the same thickness, of very loose material, consisting chiefly of fragments of rock 1 in. in diameter and smaller, with sand and clay, upon the bottom of which was a facing of about 1 in. of blue clay, the under face of which was smooth. Next came 6 ins. of mixed clay and rock fragments, below which bed-rock was found, smooth and without fissures. The total thickness of the distinctly clay seam was about 3 ins., through which a line of cleavage, indicating the movement plane, was defined clearly. The dip of the bed-rock was to the northeast, and about 1 on 4.5.

The depth below the surface was 101.4 ft., equal to Elevation (278.2).

Only a small quantity of water collected in this shaft at any time during the progress of the work, and at no time since the excavation was completed has there been in the shaft a greater depth of water than 8.5 ft., with only slight fluctuations between summer and winter.

Shaft No. 6.—The next shaft undertaken was No. 6, located near the cable track, alongside of Boring No. 5 made in 1895, and about 230 ft. south of Shaft No. 2. The elevation of the surface of the ground was (368.5), and the depth of the excavation 50 ft. The work was begun on September 17th and completed on September 29th, 1897.

For the first 12 ft. the material was a mixture of clay and fragments of basalt nearly decomposed. Then, for 27 ft., the material was found to be darker in color and harder. As a rule, the fragments of rock were small, although some pieces measured about 1 cu. ft. in volume. Next was found 7 ft. of small fragments mixed with clay, reaching a depth of 46 ft., at which point, Elevation (322.5), a seam of blue clay, a few inches thick, was found, which showed a distinct line of cleavage. The dip of the seam was eastward, 1 on 3.5. There were pebbles within 1 in. of the face of the seam, above and below. The rock below the clay seam was quite soft, but, as the excavation proceeded, rapidly grew harder, until at the depth of 4 ft. below the clay, or 50 ft. from the surface, Elevation (318.5), it became too hard to excavate without blasting, and work was discontinued.

There was some difficulty with the ventilation of this shaft, but the water caused no inconvenience whatever. At one point, near the bottom, about 40 galls. of water accumulated during one night. At no time since the shaft was completed has the water exceeded a depth of 9.5 ft., and, during the summer, the shaft is practically dry.

As has been stated, this shaft was beside Boring No. 5, the curbing for the shaft enclosing the pipe casing. As the excavation of the shaft proceeded, it became necessary to remove the pipe, which was found to have been broken near the bottom. Later, the elevation of this break was found to be (323.8), or 1.3. ft. above the clay stratum along which the movement was taking place. By the rod measurements made from the surface, as heretofore described, it had been

determined some months before that the break in the pipe and the bottom of the slide were at Elevation (324.5), or 0.7 ft. above the point where the break actually occurred and 2 ft. above the movement seam in the clay. The measurements in the shaft thus furnished a good check on the former work.

By measurements made at this shaft, it was also determined that the rate of movement at the surface and at bed-rock level had been practically the same. When Boring No. 5 was made, in 1895, the drill was broken at a depth of 47.6 ft. After spending some time fishing for the broken tool, the work was abandoned, as the engineers were then quite confident that bed-rock had been reached. Later, when Shaft No. 6 was excavated, the pipe put in as a casing for Boring No. 5 was uncovered, and the foot of the pipe was found to be about 2 ft. east of the point where the broken drill was still standing. This distance, 2 ft., corresponded almost exactly with the eastward movement which the surveys showed had taken place at the surface of the ground during the same period. After completing the shaft, a range line was established crossing it from south to north; and points were set both at the surface and at the bottom of the well. Subsequent observations of this range line showed that between October 25th, 1897, and November 22d, 1899, the total surface movement was 0.49 ft. to the eastward, while the movement at the bottom of the shaft was 0.47 ft. during the same period, a difference of only 0.02 ft.

Shaft No. 7.—Shaft No. 7 is on the roadway, a short distance west of the retaining wall at the north end of Reservoir No. 3, and alongside of Boring No. 14 *C*. The elevation of the surface was (311), and the depth of the shaft 53.5 ft.

Work was begun on September 21st, and continued until October 1st, 1897. Work was resumed on April 15th, and continued at intervals until December 25th, 1898. The total depth reached was, as stated, 53.5 ft., equal to Elevation (257.5). To make further explorations of the line of the slide, two tunnels were run in a southerly direction, one at Elevation (287) and the other from the bottom of the main shaft, Elevation (257.5). In the shaft and also in the tunnels the line of the movement seam was uncovered. In the first 14 to 15 ft. from the surface the material consisted chiefly of yellow clay containing broken rock. Next came a seam of plastic yellow clay, very tough, having a southerly inclination of about 1 on 1.3. The casing

of Boring No. 14 *C* was found broken at Elevation (296.8), or within 0.2 ft. of the clay seam at the west side of the shaft. The upper portion of the pipe had moved 0.7 ft. eastward, plainly indicating the depth at which the movement of the slide was taking place. Below the yellow clay was 27 ft. of blue, sandy clay, which was very hard when dry, as was the case in this shaft, but which, when immersed in water, would soon dissolve. Below the blue clay came about 1 ft. of tough, yellow clay, similar to that found on top of the blue clay, and having about the same southerly inclination. The depth from the surface was 43 ft. Next below, loose rock was found, with a small quantity of clay mixed, into which the shaft was carried 10.5 ft. to Elevation (257.5).

The direction of the tunnels was nearly parallel with the west line of the City Park. The upper one, at Elevation (287), cut the movement seam between the red and blue clay at 6.4 ft. from the south edge of the shaft, and entered the reddish clay mixed with loose rock, in which it continued for 70 ft., at which point the work was abandoned. In the lower tunnel, the movement seam was found in a stratum of plastic yellow clay from 1 to 1.5 ft. thick, and 52.5 ft. from the shaft. This tunnel was continued for a distance of 74 ft. The material was loose rock mixed with a small proportion of clay. The southerly dip of the movement seam passed below the tunnel level, and, in order to develop it still further, a pit was started below the bottom at a point 64 ft. from the shaft. At a depth of 8.7 ft., or Elevation (248.8), fragments of red clay on top of yellow clay were found having smooth faces, indicating the line of the slide, but the flow of water from the crevices in the loose rock was so strong that it caused the abandonment of the work at that depth. The water rose in the pit 6 ft. in one hour, but at no time did it reach the tunnel level.

Shaft No 8.—This shaft is a few feet from the margin of the old "pool," west of the cable track, and near the upper end of the sewer put in to drain the pool and adjacent ravine. The elevation of the surface was (446.4), and the depth of the excavation 63 ft. The work was begun on October 4th and completed on October 16th, 1897. The tunnel excavation was made between November 12th and December 1st, 1897. For the first 10 ft. from the surface, the excavation was through filled ground, below which came 18 ft. of blue clay containing a liberal mixture of fine sand. Next came 19 ft. of material of the same

character, and, in addition, water, which flowed at the rate of 30 galls. per hour. In places the walls of the shaft caved badly, owing to the quicksandy nature of the material; and, until the excavation had passed into more stable ground, a night crew kept the shaft pumped out and the walls dry, so as to prevent caving. Below the depth of 35 ft. there was no difficulty from this source, the material being much harder and the walls not as easily affected by the water, which still continued to percolate through the ground and into the shaft.

Between the depths of 47 to 52 ft. there was found a stratum of tough, dark blue clay mixed with small fragments of hard basalt. Below the blue clay came a 10-ft. layer of tough, plastic yellow clay, with little or no rock intermixed. At the bottom of the yellow clay, at a depth of 62.7 ft., Elevation (383.7), and just above bed-rock, there was found about 2 ins. of dark blue clay, not as plastic as the yellow, upon the top of which, between the yellow and the blue, the line of cleavage which indicated the plane of the movement could be discerned. The bed-rock dipped about 1 in 4.5 to the eastward.

From the foot of the shaft a tunnel was started in a northwesterly direction, for the purpose of developing the northern margin of the sliding land. The material in the tunnel proved to be mostly yellow clay, which was quite hard. At a distance of 22.3 ft. the tunnel cut the movement seam, which was nearly vertical. A few feet further on, large masses of rock were found, cemented and appearing to be in place, beyond which the excavation was not carried. From measurements made at the level of the tunnel, and on the surface, the inclination of the northern rim of the slide at this point was found to be 1 vertical on 0.6 horizontal.

During the last part of the work on the tunnel water collected at the rate of 50 galls. per hour. Within one week after the tunnel was completed, the water in the shaft had reached a depth of 6 ft., and, later, a depth of from 9 to 10 ft., maintaining that height, with little fluctuation, until the final draining of the shaft in 1900.

Shaft No. 9.—This shaft was started but a few feet from Boring No. 7, near the point where the cable road first touches Kingston Avenue and turns to the north along that street. The surface elevation was (460.8) and the depth 55.5 ft. Work on this shaft was begun on October 19th and completed on November 3d, 1897.

For the first 13 ft. the excavation was in yellow sandy earth. Next

came 19 ft. of blue clay with fine sand, quite quicksandy. Some difficulty was caused by the caving of the walls of the shaft. At this time the drainage from the sides of the shaft was about 30 galls. per day. For the next 21 ft. the blue clay was dryer and tougher, with less sand, and was difficult to handle. Before reaching bed-rock, at 53 ft. depth, Elevation (408), a stratum of from 6 to 8 ins. of plastic blue clay was met, in which the line of cleavage was found within 1 in. of the bottom. The movement had been taking place along this line. The slope of the seam was 1 in 2.2 to the eastward. Below the slide, the surface of the rock was decomposed and full of seams to a depth of 2.5 ft., and could be removed with a pick. At 55.5 ft., or Elevation (405.3), the rock became much harder, and work was suspended.

After passing below the slide, the quantity of water increased rapidly, coming in through the fine seams in the rock, more coming from the southwest than from elsewhere. The quantity removed before the shaft excavation was completed averaged more than 400 galls. per day.

Shafts Nos. 10 and 12.—The surface of Shaft No. 10 was at Elevation (445.8), and its depth was 22.5 ft. The surface of Shaft No. 12 was at Elevation (442.2), and its depth was 25.5 ft. To provide an adequate system of surface drainage for the water-shed of King ravine, which borders the sliding land on the north, the consulting engineer recommended the construction of a culvert to extend from the 24-in. sewer near Reservoir No. 3 westward to some point west of the cable railway, where the subterranean flow, also, could be cut off and be brought to the surface, and into the culvert.

In pursuance of this plan, Shafts Nos. 10 and 12 were excavated in the bed of this ravine, a short distance west of Kingston Avenue, for the purpose of determining the nature of the material near the upper end of the proposed culvert. By these excavations it was ascertained that the bed of the ravine, for several hundred feet west of the bridge, was composed of a mass of loose rock, mostly of large size, with gravel and sand intermingled and covering the same, through the interstices of which the water which found its way under the surface quickly disappeared. As an outcome of these explorations, it was necessary to extend the culvert several hundred feet further west than was intended originally, so as to reach a point where the rock came near to the surface.

Shaft No. 11.—When Boring No. 8 was completed, in December, 1895, the drill passed through a body of blue clay, very wet and nearly 50 ft. in depth, and soon after it was found that the inside of the pipe was filling with "blue mud." The reason for this could not be ascertained at the time, but the action was thought to be due either to a crack in the casing or to the fact that the pipe had not been driven sufficiently to make a close joint with bed-rock, thus allowing the mud and water to enter the pipe at the bottom. To gain further information regarding this deposit of quicksandy material, it was decided to sink a shaft near the boring formerly made. The location chosen finally for Shaft No. 11 was just west of the cable track, and also on the claim line, and about 40 ft. southwest from Boring No. 8. The depth of the shaft was 51 ft., and the surface Elevation (462.9). The excavation of this shaft was in progress from November 5th to 10th, and from December 2d to 8th, 1897, when a sudden inflow of water caused a suspension of the work.

For 9 ft. below the surface the material consisted of the usual clayey earth, and then came 32 ft. of blue clay similar to that found in Shaft No. 9, which was about 300 ft. to the south. Between the depths of 25 and 35 ft. the material was very soft, water collecting at the rate of 15 galls. per hour. At 41 ft. the material changed to loose rock mixed with clay, which continued for a depth of 9 ft., or to a point 50 ft. below the surface. During some nights 400 galls. of water collected. The upper surface of this stratum of loose rock and clay dipped to the north and east about 2 vertical to 1 horizontal. Between 50 and 51 ft., below which depth the excavation was not carried, the material became quite loose, and was shoveled easily. It consisted chiefly of coarse sand and loose rock, $\frac{3}{4}$ in. to 2 ins. in diameter. When work was suspended, on the evening of December 7th, the excavation had been carried into this loose material to a depth of 1 ft.

On the morning of the 8th, water was found standing in the shaft at a depth of 46.5 ft., or within 4.4 ft. of the surface of the ground. Immediately following the inflow, several attempts were made to bail out the water by hand, but all were unsuccessful. One trial of hand bailing was made between January 10th and 24th, 1898, when the removal of 77 000 galls. lowered the water level only 2.1 ft. Bailing having failed, it was next determined to try the well-boring machine

for the purpose of ascertaining the depth to bed-rock Work with this machine was begun on March 9th and completed on March 11th, 1898, and resulted in finding hard rock 17 ft. below the bottom of the shaft already excavated (or 68 ft. below the surface), the drill passing through loose rock for the entire 17 ft. The elevation of bed-rock was (395).

To test still further the volume of water in the underground reservoir connected with this shaft, operations were begun on October 17th, 1898, with the steam pumps used formerly at Shaft No. 1, and pumping was continued with regularity until November 30th. While pumping was in progress the water level was lowered somewhat, but it was found that a very slight impression was being made upon the body of water connected with the shaft. The first hour after pumping ceased the water in the shaft rose 10 ft., and measurements made a few days later showed that the water level had been lowered 6.1 ft. The estimated volume pumped was 546 700 galls. Finding that constant pumping for six weeks had lowered the water level only 6.1 ft., or less than one-seventh of the total depth, it was decided to abandon further attempts to drain the shaft by such methods. During the following winter the water stood at Elevation (458.9), or about 4 ft. below the surface of the ground.

Shaft No. 13.—This shaft was begun near the head of the slide and about 200 ft. south of Shaft No. 4. The elevation of the surface was (531.9). Work was begun on December 18th, 1897, and completed on January 6th, 1898. The depth of the shaft was 81.4 ft.

For the first 12 ft. yellow surface earth was found. Between 2 and 10 ft. a vertical crack was noticed, having a course northeast and southwest. Next, 5 ft. of blue quicksandy clay was found, followed by 15 ft. of yellow quicksandy clay in which water collected at the rate of from 20 to 40 galls. per hour. The next stratum was 20 ft. of yellow clay without sand, below which was one of 11 ft. of tough red clay, hard to work. The inflow of water was about 10 galls. per hour. The stratum of red clay was underlaid by 12 ft. of a mixture of yellow clay and broken stone, some of the clay being quite soft. From 20 to 60 galls. of water collected per hour, most of it coming from points within 20 ft. of the surface. At a depth of 73 ft. a seam in the clay was discovered, dipping to the south, about 1 on 2, and at 75.5 ft. the main slide was uncovered, with yellow clay on top of from 6 to 10 ins.

of hard blue clay. The slope was 1 on 3.5, with a slight descent to the eastward. Below the movement seam loose rock was found, which, at a depth of 81.4 ft., was too hard to pick, so that the work was abandoned. The water collected in the shaft at the rate 8 galls. per hour, and during the winter of 1898–99, it stood at a depth of 74 ft.

Shaft No. 14.—This shaft is on the south side of the "round top" and about 200 ft. west of the cable track at Shaft No. 9. The surface elevation was (506.1), and the depth of the excavation 69.3 ft. Excavation was begun on January 31st and completed on February 26th, 1898.

Below the surface, a stratum of yellowish clayey earth, 8 ft. thick, was found, followed by 11 ft. of very tough red clay. Below the red clay was found broken rock, decomposed considerably on top, but growing harder gradually as the depth increased, and having many vertical seams. For a depth of 33 ft. the fragments of rock were mostly small, some being ¼ cu. ft. in volume. Below this was a 2-ft. stratum of harder rock, the blocks being about 1 cu. ft. in volume. For the next 10 ft. the fragments were smaller, and then came 5 ft. of very fine rock, mixed with coarse sand. Below this was a stratum of clay, about 4 ins. thick, resting upon bed-rock. The usual line of cleavage in the clay was found in this instance within ½ in. of bed-rock. The slope of the bed-rock and movement seam to the southeast was about 5 ins. in 3.5 ft. horizontal.

After first penetrating the loose rock, water came in freely from all sides, but the flow was strongest from the south and east. Constant pumping was required to keep the shaft clear so that the work could proceed. The pumps were kept manned for two nights in order to finish the work. Pumping was suspended on February 26th, and in 40 hours thereafter the water in the shaft was 26 ft. deep.

Shaft No. 15.—This shaft is on the bank, about 300 ft. west of the power-house, and by the side of Boring No. 16. It was first thought that Boring No. 16 had reached bed-rock; later, when it was noticed that no bending of the pipe occurred while the surface continued to move, it was decided to sink a shaft at this point. The excavation was begun on February 28th, and completed on April 9th, 1898. The surface elevation was (347.5), and the depth of the excavation 115 ft.

Next below the surface was found a stratum of 33 ft. of yellowish sandy material, followed by 14 ft. of plastic yellow clay. Next was

found 17 ft. of tough yellow clay mixed with fragments of decomposed basalt, and below this 11 ft. of loose rock honey-combed and with a small mixture of clay, Elevation (272). For the next 16 ft. the material was of a character similar to that immediately above, with some variation in the hardness and size of the fragments of broken rock. Between the depths of 93 and 107 ft. there was found a stratum of fine broken rock varying from ¼ to 1 in. in size, and very easily picked. From 107 to 108 ft. the fragments of rock were larger, being 2 cu. ins. in volume, and the rock was very compact and hard to pick. From 108 to 109 ft. there was some clay, mixed with broken rock. At 109 ft., or Elevation (238), was found a seam in the clay showing movement, a thin layer of blue clay resting on tough red clay. At 114 ft., or Elevation (233), bed-rock was found at the west side of the shaft, with a slope to the northeast of 1 on 3. There was mixed material on the bed-rock to a depth of 1 ft. The upper 6 ins. was dark blue clay with some small stones, while the bottom 6 ins. was plastic clay, lighter in color. A plainly defined line of movement, within 2 ins. of bed-rock, was observed. At 59 ft. from the surface was found a crevice, 7 ins. wide, filled with loose material, and at this point water collected at the rate of about 30 galls. per hour. At 75 ft. the bottom of the casing for Boring No. 16 was found resting on top of a large boulder, the pipe being only about 6 ins. out of plumb. At this point the material was loose, and all water which dripped into the shaft soaked away.

The material at the bottom of the shaft, below Elevation (240), was water-tight, and, while the last of the excavation was being made, the water rose in the shaft at the rate of 4 ft. per hour. It was only after bailing and pumping for some days that the flow diminished sufficiently to permit uncovering the bed-rock.

Shaft No. 16.—The point selected for Shaft No. 16 was opposite the south end of Reservoir No. 3, and only a few feet from the west boundary of the Park. The surface elevation was (331.7), and the depth of the shaft was 97 ft. Excavation was begun on April 9th, and completed on May 17th, 1898.

For 31 ft. from the surface, the excavation was in yellowish surface clay, a small quantity of water collecting each night. From 31 to 65 ft., Elevation (267), the material was fine broken rock, which it was necessary to pick; but it was found that the walls of the shaft would cave in unless they were supported. From 65 to 82 ft., Elevation (250),

seamy rock was found, which could be removed with a pick. Some water entered the shaft from the east. From 82 to 97 ft. the excavation was still in rock, and proceeded slowly. From 95.5 to 96.5 ft. fine loose rock was found. At 97 ft., or Elevation (235), a thin stratum of blue clay was found lying on yellow clay, with bed-rock underneath. The bed-rock was honey-combed, but there were no seams. The slope was 1 on 3.5 to the east and southeast. During the night 500 galls. of water collected, coming mostly from the northeast. There were indications of the clay movement seam, but, on account of the water, the usual sample showing the line of cleavage could not be obtained.

Shaft No. 17.—This shaft was excavated at a point about 360 ft. east from the cable track, and opposite the "pool." The excavation was begun on April 19th and completed on May 20th, 1898. The surface elevation was (407.6). The depth of the shaft was 103 ft. Next below the surface was found 8 ft. of old embankment, and below this 11 ft. of yellowish, sandy clay. The next stratum was 12 ft. of loose rock mixed with clay, below which was a stratum of 1 ft. of quite solid rock. Below this, from 32.5 to 93 ft., was found a stratum of small rock, from 1 in. to 4 ins. in diameter, mixed with yellow clay in about equal proportions. The elevation of the base of this stratum was (315). From 93 to 101 ft., Elevation (307), large blocks of stone, up to 2 ft. in diameter, were found, with little clay. From 101 to 102.3 ft., was found a layer of small fragments of rock ¼ in. to 2 ins. in diameter. Between 102.3 and 103 ft., Elevation (304.6), a stratum of yellow clay on blue clay, resting on bed-rock was found. The slope was about 1 on 3 to the northeast. The movement seam was found between the yellow and the blue clay. A small quantity of water entered the shaft from the southwest.

Shaft No. 18.—Excavation for this shaft was begun on the face of the bank 210 ft. west of the west side of Reservoir No. 4, and opposite the point of greatest movement of the parapet walls. The surface elevation was (318), and the depth of the excavation 115.5 ft. Work was begun on May 18th, and the shaft and the tunnel connected with it were completed on December 16th, 1898. The following materials were found: From 0 to 15 ft., Elevation (303), yellow, sandy clay; from 15 to 38 ft., Elevation (280), very hard sand; from 38 to 80 ft., Elevation (238), loose rock, in small fragments, mixed with a small

quantity of clay. About midway in this mass there was a 2-ft. stratum of loose rock, 1 in. to 6 ins. in diameter, without clay, where the walls of the shaft tended to crumble and slide. At a depth of about 80 ft., Elevation (238), the excavation entered dark yellow or reddish clay mixed with small fragments of rock.

Movement seams in red clay, were found at 86.5 ft., Elevation (231), at the west, and at 87.5 ft., Elevation (230), at the east, side of the shaft.

At 95 ft., Elevation (223), the excavation was still in plastic red clay, when, on June 7th, 1898, a fragment of clay, having a smooth face, was loosened in the bottom of the shaft, and water flowed from the cavity with considerable force, rising in the shaft to a height of 3 ft. in 15 minutes, so that a suspension of work was necessary. After several ineffectual attempts to lower the water by hand bailing, a steam pump was set up, and, by August 22d, the flow had diminished to such an extent as to admit of deepening the shaft (through loose material, coarse sand, and stones 6 ins. in diameter and less), to Elevation (214). On August 22d, 1898, pumping was suspended. The total volume of water removed from the shaft then amounted to more than 655 000 galls.

Later in the season, it was decided to run a branch from the old drainage tunnel behind Reservoir No. 4, to Shaft No. 18, to draw off the water which continued to collect at that point. This branch was started from the old tunnel in a bed of "blue clay," and, at 42.5 ft., passed into loose rock faced with about ½ in. of yellow clay, the line of movement showing between the blue and yellow clay. The dip to the east was 1 on 3. When within 10 ft. of Shaft No. 18, the flow of water was at the rate of 75 galls. per minute, but did not continue long at that rate. After reaching and draining the shaft, the excavation was deepened below the floor of the tunnel, or to Elevation (202), when the work was suspended. The material continued to be seamy rock, through which water flowed freely, coming from the west. The total length of the branch tunnel excavated was 131 ft.

Shaft No. 19.—The point selected for this shaft was 10 ft. west of the west end of Dam No. 3. The surface elevation was (292.5). The excavation was begun on June 8th and completed on July 30th, 1898. The depth of the shaft was 60.5 ft. From the surface to a depth of 50 ft., Elevation (292.5), the upper portion of the stratum was loose rock

with which some clay was mixed, the pieces of rock in the lower portion being from 1 in. to 6 ins. in diameter, and quite compact, without admixture of clay. At Elevation (245) the shaft intersected the bottom of the drainage tunnel excavated in 1895 along the western margin of Reservoir No. 3. The sewer pipe was found to have been cracked. From 50 to 60 ft., Elevation (232.5), the material was chiefly rock, (some fragments containing 1½ cu. ft.), with little or no clay, and was difficult to excavate.

At 61 ft., Elevation (231.5), there was a strong flow of water from the southwest. Here the material changed from seamy rock, very compact, with little clay, and became quite loose. The thickness of the loose material was about 1 ft. Smooth faces on a fragment of clay which was removed indicated the movement seam at Elevation (231).

Between June 20th and July 1st, 1898, more than 390 000 galls. of water were bailed from this shaft, and, later, the excavation was deepened to Elevation (223). The rock in the bottom appeared to be in place, but was full of seams.

Shaft No. 20.—This shaft was intended simply as an opening into an old drainage tunnel, built in 1894 to drain the ravine north of Reservoir No. 3. The work of opening the shaft and cleaning out the tunnel was begun on June 30th and completed on August 24th, 1898. The end of the old tunnel was reached at a point 234 ft. from the north end of Reservoir No. 3. Later in the season, work was resumed at this point, and an extension of the tunnel was begun which was carried, finally, to the western margin of the city property, a total distance of 304 ft. from the reservoir. This work was completed on February 23d, 1899.

Shaft No. 21.—The excavation for this shaft was made in the bottom of Reservoir No. 3, opposite the foot of the buttress wall on the west slope. The elevation of the surface of the concrete lining was (259.1). The depth of the excavation was 58 ft. Excavation was begun on December 23d, 1898, and completed on January 6th, 1899. Bed-rock was found at a depth of 58 ft., at Elevation (201). The surface of the rock was very hard and uneven, without seams.

For the entire depth, the material was blue, sandy clay, very hard and dry. No water was found.

Between depths of 39 and 41 ft., Elevation (218), several small sticks of wood were found, and among the sticks the fossil tooth

mentioned previously. At 50 ft., Elevation (209), there was found a small quantity of washed gravel.

Shaft No. 22.—This shaft is opposite the buttress, and about 100 ft. west of Reservoir No. 3. The surface elevation was (305.2), and the depth 57.2 ft. The excavation was begun on January 7th and completed on January 24th, 1899. For the first 26 ft. from the surface the material was chiefly yellowish sandy clay, full of seams. From 26 to 55 ft., Elevation (250), loose rock mixed with clay was found, followed by 1 ft. of fine broken rock, at Elevation (249), lying on top of the clay movement seam which was about 3 ins. thick. The elevation was (249) at the south and (248) at the north side of the shaft. Below the plane of the movement there was ½ in. of yellow clay. The total depth of the shaft was 57.2 ft. But little water was found.

Excavation West of the Power-House.—Signs of undue pressure against the west wall of the power-house having appeared, it was decided to excavate a trench parallel with the building and extending out from the foundation for a few feet.

Work on this trench was completed on June 15th, 1898, the rock having been uncovered at Elevation (224.4). Just above the rock, blue clay was found, showing a line of cleavage, as though movement had taken place at that point, as had been suspected. The foundation of the power-house extended below the line of the slide and was immovable, although the earth adjacent to the wall was compacted very firmly.

Trench West of Dam No. 3 and North of Shaft No. 19.—On October 3d, 1898, the excavation of the trench near the west end of Dam No. 3 was completed. This trench was 14 ft. long, 3 ft. wide and 14 ft. deep, and was dug opposite a large crack in the parapet wall. The material was found to be broken rock mixed with clay, the quantity of clay diminishing toward the bottom. The mass appeared to have been broken and deranged by the pressure to which it had been subjected, but no secondary slip or line of movement was found.

A second trench, a few feet further north, and excavated a few days later, did not indicate any change in the material.

In addition to the foregoing, between February 21st and April 12th, 1898, eight 3-in. borings were made at various points on the sliding ground, ranging in depth from 42 to 121 ft., and aggregating a total depth of 655 ft., or 677 ft. including the work done in the bottom of Shaft No. 11. The contract price per foot for these holes was $0.60 in

earth and $1.25 in rock, the actual cost of the necessary casing was to be paid for at an extra price, and, in addition, wood and water for the engine was to be furnished to the contractor without charge. The total cost of the work amounted to $764.27, approximately, or $1.13 per linear foot.

TABLE 1.—Summary of Borings Made During 1895 and 1896.

No. of boring.	When begun.	When completed.	Surface elevation.	Depth of boring, in feet.
	1895.	1895.		
1	Sept. 27	Dec. 10	(223.7)	68.0
2	" 28	Oct. 15	(237.3)	72.0
3	" 30	Nov. 16	(264.8)	83.5
4	Oct. 30	" 6	(333.6)	106.8
5	Nov. 8	" 11	(369.0)	47.6
6	" 16	" 23	(409.8)	79.6
7	" 25	" 29	(460.8)	59.0
8	Dec. 2	Dec. 3	(461.2)	72.0
9	" 3	" 9	(458.4)	91.4
10	" 12	" 13	(458.9)	34.0
11	" 16	" 21	(426.0)	106.0
12	" 24	" 26	(397.5)	94.5
13	" 27	" 30	(344.3)	76.0
	1896.	1896.		
14 *C*	Jan. 20	Jan. 28	(311.3)	76.0
15	" 12	" 13	(340.4)	32.0
16	" 14	" 18	(347.9)	79.0
17	" 29	" 31	(313.8)	64.2
18 *B*	Feb. 4	Feb. 7	(311.7)	43.5
19	" 8	" 12	(303.5)	25.2
20	" 12	" 24	(296.9)	43.0
21 *B*	" 24	Mar. 5	(292.4)	77.5
22	Mar. 6	" 12	(298.4)	95.0
23	" 13	" 20	(305.9)	57.6
24	" 21	" 25	(308.8)	88.0
25	" 25	" 31	(308.9)	44.0
Total, from September 27th, 1895, to March 31st, 1896				1 710.4

TABLE 2.—Expenditure on Account of Borings Made During 1895 and 1896.

Date.	Contract account.	Material account.	Extra labor account. (Estimated.)	Total.
1895.				
Sept. and Oct		$31.33	$150.00	$181.33
Nov	$485.33		50.00	535.33
Dec	592.40	5.00	50.00	647.40
1896.				
Jan	349.87	104.87	50.00	504.74
Feb	383.13	78.09	50.00	511.22
Mar	366.25	78.55	49.50	494.30
Total	$2 176.98	$297.84	$399.50	$2 874.32

Cost of the Work: Linear feet of borings, 1 710; average cost per linear foot, $1.68.

TABLE 3.—SUMMARY OF BORINGS MADE DURING 1898.

No. of boring.	When begun.	When completed.	Surface elevation.	Depth of boring, in feet.
	1898.	1898.		
26	Feb. 1	Feb. 8	(449.4)	80.0
27	" 8	Mar. 9	(446.3)	75.5
28	" 22	Feb. 24	(430.2)	98.5
29	" 25	" 28	(425.5)	111.8
30	Mar. 12	Mar. 19	(403.0)	75.5
31	" 21	" 29	(368.2)	51.0
32	" 29	Apl. 8	(407.3)	121.0
33	Apl. 11	" 12	(346.4)	42.0
Shaft No. 11	Mar. 9	Mar. 11	.....	22.0
Total, from February 1st, 1898, to March 11th, 1898				677.3

The explorations required to determine the depth to bed-rock, and the character of the overlying materials, described in the foregoing pages, were completed early in 1899. While the excavation of the shafts was in progress, observations of the water levels in various shafts and borings were taken at frequent intervals, in the hope of finding some correspondence in such levels, but, with one or two exceptions, no indications were discovered showing any connection between the underground waters at the several shafts.

The results of the shaft and tunnel excavations, the measurement of water levels and the rate of movement from month to month, were communicated to the consulting engineer as rapidly as the data were gathered.

TABLE 4.—EXPENDITURES ON ACCOUNT OF BORINGS MADE DURING 1898.

Date.	Contract account.	Extra labor account. (Estimated.)	Total.
1898.			
Feb.	$227.50	$40.00	$267.50
March	365.00	40.00	405.00
April	71.77	20.00	91.77
Total	$664.27	$100.00	$764.27

Cost of the Work: Linear feet of borings, 667; average cost per linear foot, $1.13.

TABLE 5.—Summary of Shaft and Tunnel Work, Completed During the Years 1897, 1898 and 1899.

No. of shaft.	When begun.	When completed.	Surface elevation.	Bed-rock elevation.	Depth of slide, in feet.	Depth of shaft, in feet.	Length of tunnel, in feet.
1	July 19, 1897	Jan. 1, 1898	(435.0)	(360.0)	75.0	75.0	.....
2	" 19, 1897	Aug. 20, 1897	(408.0)	(297.0)	111.0	112.0	.....
3	Aug. 9, 1897	" 10, 1897	(467.5)	.....	.....	10.0	.....
4	" 11, 1897	" 23, 1897	(530.5)	(457.5)	73.0	73.0	32
5	" 23, 1897	Sept. 21, 1897	(379.6)	(278.2)	101.4	101.4	.....
6	Sept. 17, 1897	" 29, 1897	(368.5)	(322.5)	48.0	50.0	.....
7	" 21, 1897	Dec. 23, 1898	(311.0)	.....	*15.0	53.5	1-74 1-70
8	Oct. 4, 1897	Oct. 16, 1897	(446.4)	(383.4)	62.7	63.0	38
9	" 19, 1897	Nov. 3, 1897	(460.8)	(408.0)	52.8	55.5	.....
10	Nov. 2, 1897	" 5, 1897	(445.8)	.....	.....	22.5	.....
11	" 5, 1897	Dec. 8, 1897	(462.9)	(395.0)	67.9	51 + boring 22 ft.	.....
12	" 5, 1897	Nov. 27, 1897	(442.2)	.....	.....	25.5	39
13	Dec. 18, 1897	Jan. 6, 1898	(531.9)	(454.0)	75.9	81.5	.....
14	Jan. 31, 1898	Feb. 26, 1898	(506.1)	(437.1)	69.0	69.0	.....
15	Feb. 28, 1898	April 9, 1898	(347.4)	(232.6)	109.0	115.0	.....
16	April 9, 1898	May 17, 1898	(331.7)	(234.7)	96.5	98.0	.....
17	" 20, 1898	" 20, 1898	(407.6)	(304.6)	102.0	103.0	.....
18	May 18, 1898	Dec. 16, 1898	(318.1)	(223.0)	86.5	115.5	131
19	June 8, 1898	July 30, 1898	(292.5)	(231.0)	60.5	69.5	.....
20	" 30, 1898	Feb. 23, 1899	(314.8)	.....	.....	18.0	70
21	Dec. 23, 1898	Jan. 6, 1899	(259.1)	(201.1)	.....	58.0	.....
22	Jan. 7, 1899	" 24, 1899	(305.2)	(248.0)	56.2	57.2	.....
Trench W. of power-house	.....	June 15, 1898	.....	(224.0)	.....	6.0	.....
Trench W. of Dam No. 3.	.....	Oct. 3, 1898	.....	.....	.....	14.0	.....
Totals					1 260.4	1 497.1	454

Average depth of slide, $\frac{1\,245.4}{16} = 77.8$ ft.

* Not included in average of depth of slide.

Summarizing, in a general way, the results of the surveys and examinations, then completed, it may be said that, after a careful study of all the facts collated, the following conclusions were reached:

(1) The approximate length of the sliding ground was 1 700 ft.

(2) The approximate width of the sliding ground along the west margin of reservoirs was 1 100 ft.

(3) The approximate width of the sliding ground at its west end was 400 ft.

(4) The approximate surface area of the sliding ground was 29.27 acres.

(5) The approximate bed-rock area of the sliding ground was 23.65 acres.

(6) The approximate area of the water-shed, which includes the western end of the sliding tract, was 26.46 acres.

(7) The minimum depth of the body of the slide (Shaft No. 6) was 46 ft.

(8) The maximum depth of the body of the slide (Shaft No. 2) was 112 ft.

(9) The average depth of the slide at sixteen shafts in the main body of the slide was 77.8 ft.

(10) The approximate volume of the sliding mass was 3 400 000 cu. yds.

(11) The approximate weight of the sliding mass was 4 600 000 tons.

TABLE 6.—Expenditure on Account of Shafts and Tunnels, During 1897, 1898 and 1899.

Date.	Labor.	Pumping.	Materials.	Total cost.
July, 1897	$131.00	.....	$64.60	$195.60
Aug. "	490.80	.....	177.22	668.02
Sept. "	624.25	.....	71.38	695.63
Oct. "	206.65	$181.00	93.94	481.59
Nov. "	441.80	192.00	52.60	686.40
Dec. "	306.95	174.25	66.24	547.44
Jan. 1898	112.00	152.90	12.86	277.76
Feb. "	190.25	43.40	28.43	262.08
Mar. "	195.75	.....	27.09	222.84
Apr. "	256.05	.....	43.70	299.75
May "	319.00	.....	27.58	346.58
June "	348.10	16.75	64.68	429.53
July "	321.10	62.50	61.55	445.15
Aug. "	178.25	48.75	27.41	254.41
Sept. "	177.30	.....	39.87	217.17
Oct. "	399.15	47.50	183.90	630.55
Nov. "	423.50	65.00	63.95	552.45
Dec. "	358.00	.....	.....	358.00
Jan. 1899	192.05	.....	62.05	254.10
Feb. "	316.60	.....	6.00	322.60
Mar. "	77.15	.....	.....	77.15
Total	$6 065.70	$984.05	$1 175.05	$8 224.80

Estimate of work done:

Shaft excavation.....1 497 lin. ft.
Tunnel excavation: 454 lin. ft., equivalent to
shaft excavation, 454 × 2, or ..... 908 " "

Total estimated shaft excavation.....2 405 " "
Cost per linear foot $3.42
Laborers' wages were $1.75 per day of 10 hours.

(12) With each returning dry season there was a marked lessening of the movement, followed by an increase during the winter, indicating an intimate relation between the volume of monthly rainfall and the rate of movement, as shown by Table 7, prepared in January, 1899, and covering the surveys completed to that date.

TABLE 7.

Time of Observation.	Total rainfall, in inches.	Total Movement, in Feet.		Average maximum movement per month, in feet.
		Minimum.	Maximum.	
Dec., 1895, to May, 1896, inclusive..	27.58	1.09	1.30	0.22
June, 1896, to Nov., 1896, " ..	18.08	0.25	0.45	0.08
Dec., 1896, to May, 1897, " ..	21.74	0.71	0.84	0.15
June, 1897, to Nov., 1897, " ..	18.69	0.09	0.14	0.02
Dec., 1897, to May, 1898, " ..	25.33	0.05	0.15	0.03
June, 1898, to Nov., 1898, " ..	13.12	0.00	0.03	0.01

The readings are the minimum and maximum average movement at eight different points along the center of the slide, between Reservoir No. 4 and the "round top."

The figures denoting the rainfall have been compiled from the reports of the United States Weather Bureau. The observations were taken at the Weather Bureau office, in the City of Portland, 1½ miles distant from the sliding land tract. Elevation about (150). It may also be stated that the six-month period following the last one given in the table showed a very marked increase in the rate of the movement.

(13) The surveys thus far made do not determine that the movement is at a uniform rate at the center and along the sides of the sliding ground, but probably there is little difference.

(14) Apparently, the movement at the surface and on the bed-rock are uniform.

(15) Apparently, there is a slight southerly movement taking place near the west end of the slide and a northerly movement as well at the central portion, the bed-rock being slightly inclined in these directions as well as to the eastward.

The inclination of the bed-rock at different points on the slide is shown by Table 8.

(16) The uniformity of the movement of the slide is indicated by the absence of surface cracks across the central portions of the sliding tract.

TABLE 8.—BED-ROCK ELEVATIONS AND SLOPES.

No. of shaft.	Elevation of bed-rock.	No. of shaft.	Elevation of bed-rock.	Distance between shafts, in feet.	Difference in elevation, in feet.	Slope of bed-rock.	Direction of slope of bed-rock.
4	(458)	13	(454)	214	4	1 on 53.5	S.
13	(454)	14	(437)	234	17	1 " 13.8	E.
14	(437)	9	(408)	198	29	1 " 6.9	E.
9	(408)	11	(395)	250	13	1 " 19.2	N.
11	(395)	8	(383)	298	12	1 " 24.8	N.
4	(458)	8	(383)	518	75	1 " 6.9	N. E.
11	(395)	1	(360)	268	35	1 " 7.6	E.
1	(360)	2	(297)	248	63	1 " 3.9	E.
8	(388)	17	(305)	420	78	1 " 5.4	E.
17	(305)	16	(235)	362	70	1 " 5.2	E.
17	(305)	7	(Tunnel) (249)	466	56	1 " 8.3	N. E.
7	(Tunnel) (249)	19	(231)	392	18	1 " 21.8	S. E.
19	(231)	Power-house.	(224)	194	7	1 " 27.7	S. E.
2	(297)	15	(233) (Mo'm't)	252	64	1 " 3.9	E.
2	(297)	18	(231)	388	66	1 " 5.9	E.
6	(323)	18	(231)	424	92	1 " 4.6	N. E.
15	(233)	18	(231)	170	2	1 " 85.0	S. E.
19	(231)	18	(231)	432	0	1 " 0.0	S.
2	(297)	5	(278)	334	17	1 " 19.6	N.
16	(235)	19	(231)	196	4	1 " 49.0	S. E.
9	(408)	6	(323)	510	85	1 " 6.0	E.
17	(305)	5	(278)	166	27	1 " 6.1	E.
5	(278)	16	(235)	298	43	1 " 5.3	N. E.

(17) That the movement originated in the unstable mass near the western end of the sliding ground is indicated by the condition of the joints in the rails of the cable road where running east and west. When first examined, in September, 1895, these joints were all tightly closed. The rails were also distorted at the angle just east of Shaft No. 6, as shown in Fig. 2, Plate XIX. These conditions seem to point to the conclusion that the pressure was mainly from above and that there had been no drawing away of the foot of the slope at the reservoirs; or, in other words, that the movement did not originate at the reservoirs.

(18) The breaks in the lining of Reservoirs Nos. 3 and 4, and also at the buttress built against the lining of Reservoir No. 3, have been caused by the movement of the slide, and are not due to local slips, as at first supposed.

(19) From the shaft excavations, the character of the materials forming the mass of the slide has been determined to be largely of broken rock of small size mixed with clay. Across the central portion of the slide, in the vicinity of Shaft No. 1, the clay and fine

material predominate, forming a dike measurably impervious to water. Eastward from that point, the material contains less clay, and does not hold a large quantity of water.

To the west of Shaft No. 2, and also near the head of the slide, the excavations developed at some points quite a body of clay. Between Shafts Nos. 1 and 11, and still further west, water pockets of considerable magnitude have been found. From the pocket in connection with Shaft No. 1 more than 3 900 000 galls. of water were pumped before the supply was exhausted; and, from the one connected with Shaft No. 11, about 500 000 galls. were pumped before work was abandoned at that point, this effort having lowered the water but a few feet.

(20) As a rule, there seemed to be no direct connection between the different bodies of underground water, or between the several shafts and borings.

(21) The several bodies of water discovered were all in close contact with the clay stratum forming the bed of the slide, and, undoubtedly, have had much to do with the continuance of the movement. This is shown by the decrease in the movement which occurred between December, 1897, and November, 1898, during which period the underground reservoir connected with Shaft No. 1 was drained by pumping. Fig. 4 is a profile showing the elevation of the water standing in the shafts.

(22) Comparing the rate of movement of the slide at different seasons of the year with the volume of the rainfall during the same period, and noting also that a greatly reduced motion ensued during the time that the drainage of Shaft No. 1 was in progress, succeeded by an increased movement during the following winter when the shaft was again filled, it seems reasonable to conclude that the movement was largely if not entirely due to the presence of water in the underground reservoirs. It follows naturally, therefore, that the permanent removal of underground waters would produce a permanent cessation of the movement.

The conclusions of the consulting engineer were practically in accord with the foregoing statements. His report also contained interesting references to the probable origin of the slide, as well as recommendations regarding the best method of overcoming the difficulty.

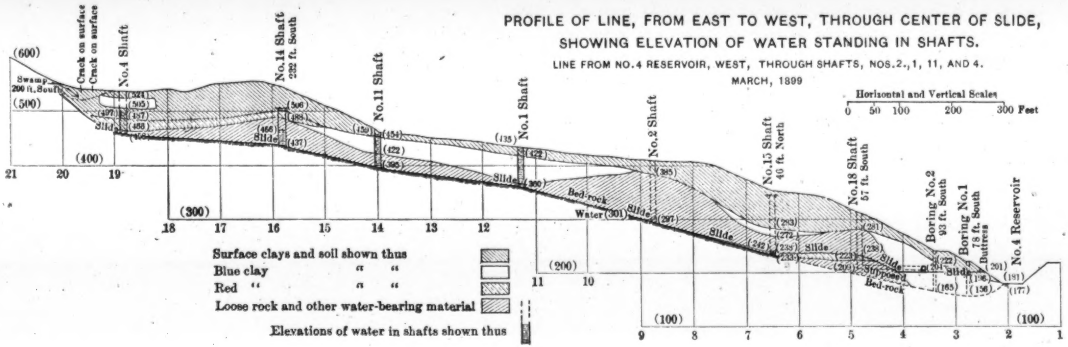

PROFILE OF LINE, FROM EAST TO WEST, THROUGH CENTER OF SLIDE,
SHOWING ELEVATION OF WATER STANDING IN SHAFTS.
LINE FROM NO.4 RESERVOIR, WEST, THROUGH SHAFTS, NOS.2., 1, 11, AND 4.
MARCH, 1899
Horizontal and Vertical Scales
0 50 100 200 300 Feet
Swamp 200 ft. South
Crack on surface
Crack on surface
No.4 Shaft
No.14 Shaft 232 ft. South
No.11 Shaft
No.1 Shaft
No.2 Shaft
No.13 Shaft 40 ft. North
No.18 Shaft 57 ft. South
Boring No.2 93 ft. South
Boring No.1 78 ft. South
Dutfres
No.4 Reservoir
Slide
Bed-rock
Water (301)
Surface clays and soil shown thus
Blue clay " "
Red " " "
Loose rock and other water-bearing material
Elevations of water in shafts shown thus

FIG. 4.

Before proceeding to discuss the remedy proposed by the consulting engineer, and the action taken subsequently in connection therewith, mention should be made of a matter which at one time overshadowed all other interests completely, threatening as it did an enormous property loss to the city.

During the latter part of the year 1897, after it became known that the Water Committee had entered upon an examination of the sliding-land question with the view of determining the best method of restoration, the owners of about five-sixths of the moving ground instituted a suit against the city for damages to their property. They claimed that the excavation of the reservoirs, in the autumn of 1893 and in the spring and summer of 1894, was the cause of the movement, and in consequence of this movement their property had been damaged to an amount aggregating $439 000.

Two of these suits were filed within a few months of each other. The first was filed on February 16th, 1898, by the King Real Estate Association, claiming damages to land, $169 000, and for injury to the cable railway in which they were interested, $100 000, making a total of $269 000. The second suit was begun on May 2d, 1898, by L. F. Grover *et al.* who claimed damages to 22.5 acres of land, at $5 000 per acre, or $110 000, and for damages to the cable railway in which they were interested, $60 000, making a total of $170 000.

The value of these tracts of land, as given on the assessment roll for the year 1897, was as follows:

King Real Estate Association, 97 acres.....	$33 000
Grover *et al.* West End Addition, 51 acres.....	28 100
Grover *et al.* West End Addition, 77 "	23 000
Total assessed valuation.....	$84 100

The claim made by the plaintiffs in these suits, that the excavation of the reservoirs was the primary cause of the movement of the adjacent hillside, was not regarded by the Water Committee as tenable, and it was contested strongly. The suits were fought by the city attorney at every stage. In consequence of these dilatory proceedings, the issues were not fully made up and the trial begun until November 7th, 1899.

The report of the consulting engineer had been submitted to the Water Committee during the preceding March, but had been with-

held from publication, on account of the damage suits which had been instituted, and in order not to appear to prejudge the case in advance of the trial. It was also desired that the conclusions of the report should be presented first in the form of testimony for the defense upon the trial then in prospect.

During the year 1899, the time of the engineers was largely engrossed in the study of the sliding-land data already gathered, and in preparing maps, plans, profiles and exhibits of various kinds for use in the coming trial, or to illustrate some new phase of the problem in hand.

The trial, which was begun on November 7th, 1899, was of exceeding interest, and importance, as well, both on account of the novelty of the conditions leading to its inception and the magnitude of the pecuniary interests involved. The case was studied closely by the city attorney, who was early impressed by the fact that the cable road had been in successful operation for nearly a year prior to the beginning of reservoir construction, and, as far as then known, there had been no difficulty in its operation. This was regarded as a strong point for the plaintiff, and as indicating that the reservoir excavation was the originating cause of the movement.

The investigations made by the engineers seemed to point conclusively to the fact that the movement of the slide must have been in progress for some years, at least intermittently, but it was impossible to determine when the movement began. Owing to the fact that the hillside was covered originally with a dense forest of firs and cedars, with a smaller growth of vine, maple and alder, of which it had been cleared but a short time prior to the construction of the cable road, it would have been exceedingly difficult if not impossible to determine the existence of any movement from a simple inspection of the ground, especially when the monthly or yearly change was as slight as it is known to have been in this instance. In further explanation of the difficulties attending such observations, it may be said that when reservoir construction was begun, the banks of the ravine in which the reservoirs are located were covered with fallen timber and dense brush, and the upper or western third of the sliding tract is, to this day, covered with brush, which, during the summer months, effectually shields the ground from observation, making imperative the use of instruments in order to gain even an approximate measurement of any movement which may be taking place.

When the facts supporting the theory of "an ancient slide" were placed in the hands of the city attorney, he at once began a campaign of investigation and search to determine the location of the men employed in surveying, building and operating the cable road during the two years that elapsed between the time that the work was begun and the final abandonment of the enterprise. The search continued for months, and, when the day of trial came, the mass of testimony which had been accumulated (all tending to substantiate the claim that the movement of the hillside was in progress when the cable road was completed, a year prior to the commencement of reservoir construction), must have been a surprise to the plaintiffs. So many witnesses were offered to testify to a movement of the cable track at certain points that the Court declined to hear them all, considering that fact established beyond question. The witnesses who testified on these points included the contractor who built the line, and the inspector and car men who operated the road after it was built, all of whom testified to the difficulties experienced in maintaining bridge bents in a vertical position on the east and west line, and the alignment of the cable conduit and rails where they crossed the slide from north to south. The contractor also recounted some of his experiences in forcing the track back into line, so as to admit of cars passing in safety, and to insure its acceptance by the company.

The engineers engaged as expert witnesses for the city formulated the theory of an "ancient slide" and showed the depth and character of the material forming the mass of the slide, as well as the inclination of the bed-rock and the presence of a stratum of plastic clay, with an abundance of water stored in the interstices of the rock in immediate connection therewith, all supported by the data to which reference has been made herein. They were also fortified by a number of plans and "exhibits" prepared especially for use on the trial, to which it may be interesting to refer briefly.

The principal one of these exhibits, and in many respects the most valuable of all, was a plaster-of-paris model of the reservoirs and the sliding ground, including also the water-shed surrounding the head of the "slide," made on a scale of 1:600, which was prepared by the Water Committee's engineers. This model showed the original ravines in which the reservoirs are located and the volume of the material removed in excavating the reservoir basins, as well as the location and

depth of the ravines which were filled by the property owners, without adequate under drainage. The mass of the slide was also cast in a separate block which could easily be removed so as to show the depth of the slide and the inclination of the bed-rock on which the movement was then taking place. From this model was shown the relative insignificance of the volume of the material removed in excavating the reservoir basins, which amounted to only about 3% of the mass of the slide, and the improbability that the removal of this small volume could have caused a movement of the slide of the magnitude known to exist.

Another interesting exhibit used upon the trial was a diagram showing a comparison of the volume of the rainfall and the movement of the slide for corresponding months, from January, 1895, to the date of the trial. This diagram has since been extended so as to cover the entire period during which the surveys have been continued. This diagram is shown in Plate XXV.

The measurement of rainfall was taken from the reports of the United States Weather Bureau in Portland. It is to be noted that while the amount of precipitation is extremely large for some months, notably 13.12 ins. for November, 1896, the average for any one season rarely exceeds 44 ins.

The movement, as recorded on the diagram, is the average of fourteen points observed at intervals along the central portion of the sliding ground, where the maximum movement was supposed to have taken place, from September, 1895, to December, 1899. Since the latter date the average of a large number of points has been taken, two hundred and sixty points having been established and observed regularly. The average of the readings at fifty-one of these points, along a central belt 100 ft. wide, has been used in plotting the diagram of movement for 1900–1903. Merely casual inspection and comparison will serve to show the apparently close relation which exists between the volume of the rainfall during what may be called the wet and dry seasons of the year—say, from December to May, inclusive, for the former, and from June to November, inclusive, for the latter —and the movement of the slide during the same period. It will be seen from the diagram that with each recurring dry season there was a corresponding cessation of the movement, and that with the beginning of the winter rains the movement increased. It is to be

noted, also, that, in each instance, an interval of about one month elapsed between a change in the volume of rainfall and the corresponding change in the rate of movement.

An inspection of the diagram will show at once one irregularity in the yearly movement, namely, a marked increase in the monthly movement during the winter months is followed usually by a corresponding diminution with the return of dry weather. The explanation of this irregularity in the movement, which occurred during the years 1897 and 1898, forms one of the most interesting of the developments which have occurred since the study of this problem was begun.

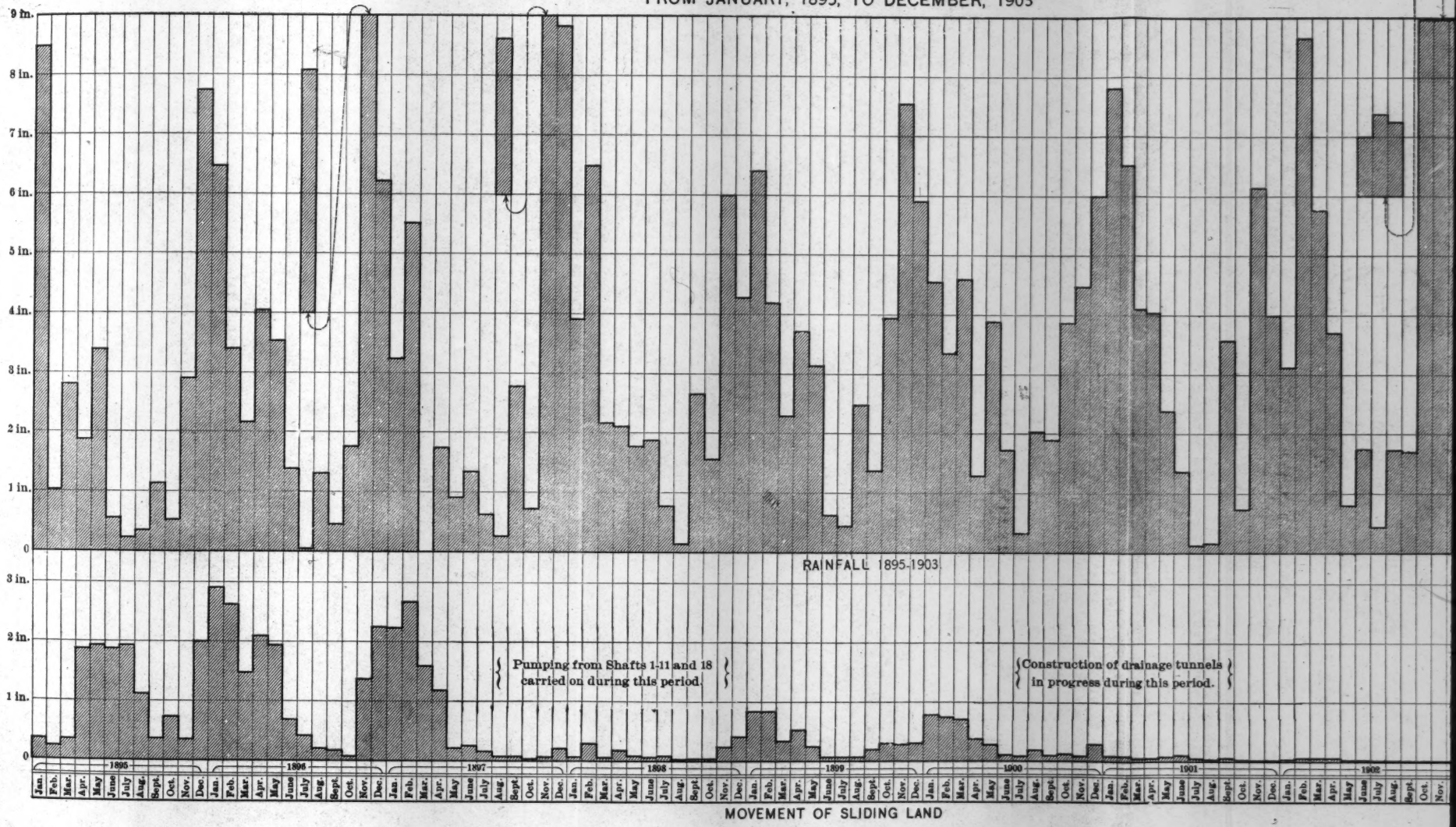
It will be recalled that while the excavation of Shaft No. 1 was in progress considerable water was found, the draining of which was accomplished by deep-well pumps. The draining of this well was in progress from August 19th, 1897, until January 31st, 1898, the total volume of water removed during that period being more than 3 900 000 galls. Even then, the underground reservoir had not been drained thoroughly, for, when pumping ceased, the water rose in the shaft to a height of several feet.

By observing the diagram it will be noted that when pumping from this shaft was in progress, the movement did not increase as rapidly during the winter months as it had during former seasons, although there was a slight increase from August to February when the maximum for that year was reached. The movement for February, 1898, was only 0.30 in., as compared with 2.69 ins. for February, 1897, and 0.85 in. for February, 1899. Perhaps this irregularity can be shown best by comparing the total movement for the entire season preceding with the total movement for the season following the drainage operations. From May, 1896, to May, 1897, the total movement was 13.10 ins. From May, 1897, to May, 1898, during a portion of which period pumping was in progress, the total movement amounted to only 1.57 ins.; and this was followed by an increase to 3.69 ins. during the year beginning May, 1898, and ending May, 1899.

It is probable that the decreased movement noted during the year 1898 was due in part to the pumping of water from Shaft No. 18, in July and August, amounting to 650 000 galls., and from Shaft No. 11, in October and November, amounting to 540 000 galls., but it is believed to have been due principally to the thorough drainage of the

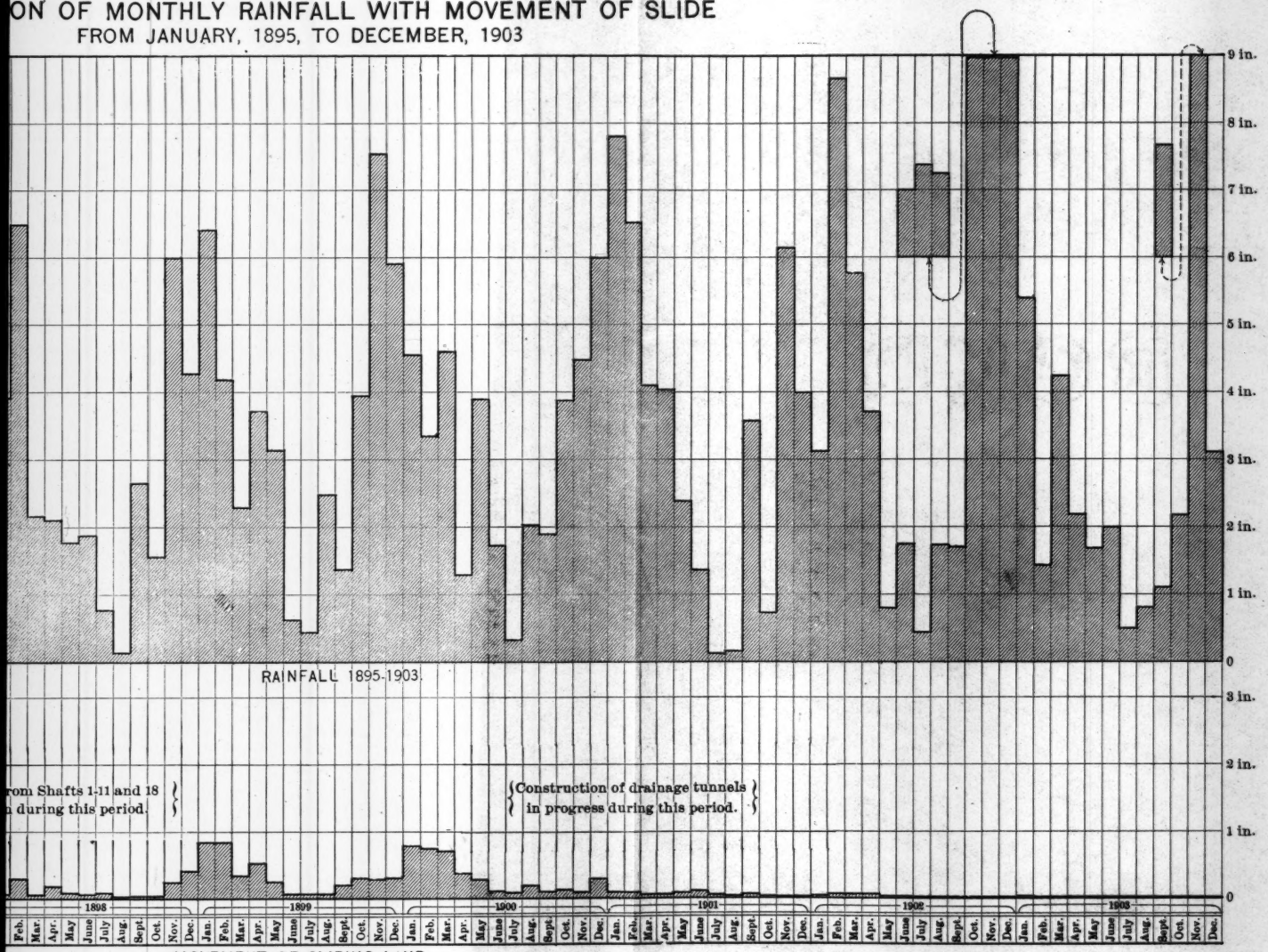
COMPARISON OF MONTHLY RAINFALL WITH MOVEMENT OF SLIDE

FROM JANUARY, 1895, TO DECEMBER, 1903



ON OF MONTHLY RAINFALL WITH MOVEMENT OF SLIDE

FROM JANUARY, 1895, TO DECEMBER, 1903



MOVEMENT OF SLIDING LAND

underground reservoir connected with Shaft No. 1. As has been stated, the drainage of this body of water was practically completed in January, 1898, pumping having been suspended on the 31st of that month, and the water was thereafter allowed to accumulate, the rise in the shaft indicating the rate at which the reservoir was being filled. This rate was noted carefully, and it was observed that the underground reservoir was not filled to its original height until the end of December of that year.

During the period when this underground reservoir was being drained, and for some months thereafter, the movement was at a greatly diminished rate. When the underground reservoir was again filled the rate of movement began to increase until it was more than double that of the preceding year, and this uniformity of rise and fall was continued for the two seasons following.

It was this seeming coincidence of the cessation of the movement with the draining of the underground reservoir at Shaft No. 1 which first changed conviction into certainty that the cause of the slide was to be found in the underground water stored in various portions of the sliding ground, and hence it followed that a remedy for the difficulty was to inaugurate a thorough system of drainage.

This diagram was one of the most important and convincing "exhibits" introduced by the Water Committee in defending the suit, as it showed, clearly, that the movement depended more, and chiefly, upon the volume of water falling upon the surface and stored in underground reservoirs in the interstices of the broken rock forming a large portion of the mass of the slide, than upon the removal of a small fragment of earth from the toe of the slope. In addition to the foregoing, a number of cross-sections of the sliding ground, and tables showing the results of surveys, etc., were introduced at the trial. The cross-sections are shown on Plate XXII.

The trial of the damage suit of the King Real Estate Association, the first one instituted, was begun on November 9th, and was terminated on November 28th, 1899, in a verdict in favor of the city.

The expert witnesses who testified for the plaintiff were Arthur L. Adams, M. Am. Soc. C. E.; Franklin Riffle, M. Am. Soc. C. E., and W. A. Crondahl, C. E. Those who testified for the city were Col. G. H. Mendell, M. Am. Soc. C. E., Consulting Engineer (since deceased); George L. Dillman, M. Am. Soc. C. E.; F. I. Fuller, M. Am. Soc. C. E.,

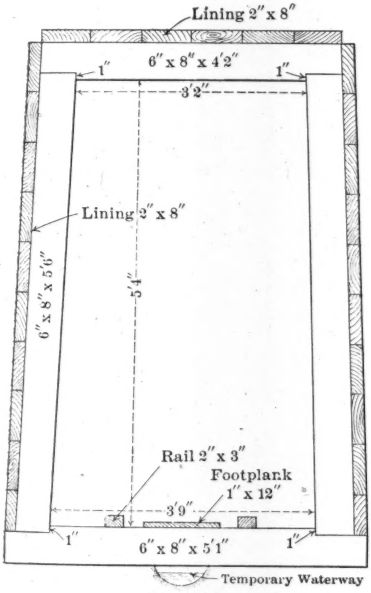
and J. A. Hurlbut, C. E. The testimony of these witnesses was of an exceedingly interesting character, but special reference to it at this time is omitted purposely, in the hope that some of these gentlemen will participate in the discussion of this paper, giving the results of their investigations and conclusions.

As a result of this verdict the second suit was not pressed for trial, although an attempt was made by the city to have it set for an early date. In a few weeks, however, the several owners of the sliding land made overtures to the Water Committee to sell to the city a large tract, nearly 60 acres, including the sliding land and surrounding water-shed, for a sum which was regarded as reasonable, and the purchase of this land was completed a few months later. Thus the Water Committee acquired control of all the lands adjacent to the reservoirs, and sloping thereto, and hence will be able to eliminate all danger of complications which might arise through occupation of the premises by private owners. This tract, ultimately, will be added to the present City Park, which it adjoins, making an area of more than 100 acres available for park purposes.

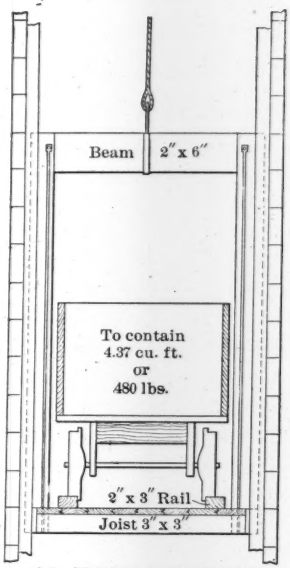
The purchase of the sliding-land tract by the city was concluded in May, 1900, the terms arranged with the property owners including an agreement for the construction of a 24-in. sewer to convey the surface drainage from the upper portion of the King ravine water-shed into the Washington Street sewer, north of the reservoirs, this sewer to be completed prior to October 1st, 1901. The drainage from this ravine, originally, was provided for by a 24-in. sewer which passed near the north end of Reservoir No. 3, and thence through the Park to the main sewer in Washington Street. The new sewer was proposed as an additional safeguard.

During June, 1900, the writer presented to the Water Committee a statement of the work then accomplished and the reasons for believing that the slide could be stopped by a thorough system of under-drainage along bed-rock, as recommended by the consulting engineer and supported by the testimony of the engineers engaged as expert witnesses upon the trial of the King damage suit, in behalf of the plaintiff as well as the defendant, and requested authority to undertake the immediate construction of the drainage tunnels in question. This report was accompanied by a map showing the main drainage tunnels proposed. The plan proposed by the consulting engineer called for

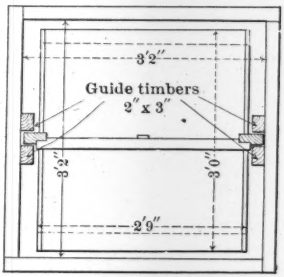
DETAILS OF DRAINAGE TUNNEL, CAR AND HOIST.



END VIEW



ELEVATION OF CAGE AND CAR



PLAN OF SHAFT AND CAGE

FIG. 5.

the construction of the main tunnel leading west from Reservoir No. 4 to the line of the old cable track on Kingston Avenue, and left the question of branches, or the further extension of the main tunnel, to be determined as the work progressed. Upon this presentation, the writer was directed by the Water Committee to begin the work of tunnel construction at once. All preliminary arrangements were soon made, and the work of excavation for the drainage tunnels was begun on July 2d, 1900.

The size of the tunnel was fixed at the smallest dimensions that would conveniently answer, the inside dimensions being 3 ft. 2 ins. wide on top, 3 ft. 9 ins. wide at the bottom, with a clear height of 5 ft. 4 ins. The timbers used were 6 x 8-in. spaced at 3-ft. centers, with 2 x 8-in. lagging. The required excavation approximated 1.25 cu. yds. per linear foot of tunnel. (See Fig. 5.)

The plan adopted for the prosecution of the work called for the commencement of the excavation at the lower end of the tunnel at the bottom of Reservoir No. 4, and also at one or more of the exploration shafts which had been opened to bed-rock two years before, the line adopted for the main tunnel passing through Shafts Nos. 18, 2 and 1 to Shaft No. 11 at the cable road. These shafts were made 3 ft. 2 ins. square, in the clear. To expedite the work at these shafts, a small hoist was constructed to be operated by a steam engine, similar to those in use by builders. This hoist, necessarily, was of rather diminutive size, the platform being only 25 ins. wide in the clear between the side timbers, and about 30 ins. long. Several small dump cars, of such size as could be carried safely upon the platform of the hoist, were made, the capacity of the box being about 5 cu. ft. The gauge of the track was 18 ins. The wheels were 9 ins. in diameter. These cars were run on a strap-iron track laid on 2 x 4-in. stringers on the floor of the tunnel, and, when filled, were pushed by hand to the foot of the shaft and thence hoisted to the surface and dumped. In most cases the grade of the tunnel was sufficient to take the car from the heading to the shaft by gravity, the empty cars being pushed back by hand. This arrangement proved to be quite effective and economical.

To expedite the work, day and night crews were employed, for a time, at two different headings, until the tunnel had been completed as far as Shaft No. 2. From that point only one heading was worked, although day and night crews were employed, each working a ten-hour shift.

Comparatively little water was encountered during the early stages of the work, and it was not until October 30th, when the tunnel reached the vicinity of Shaft No. 1, a distance of 890 ft. from Reservoir No. 4, that water was found in any considerable quantity. At this point, the water stored in Shaft No. 1, then standing at a depth of 70 ft., was drained away. A few days later the tunnel was carried to the foot of the shaft, 912 ft. from Reservoir No. 4, and connection was made with the water-bearing stratum from which the shaft had been supplied. For a number of hours the discharge of water from this source was so strong that it overflowed the box gutters prepared for it, and caused the suspension of all work. The flow diminished somewhat within an hour or two, but it was estimated that approximately 1 800 000 galls. were drained away within the next twenty days, by which time the flow had diminished to about 25 000 galls. per day.

About the middle of January, 1901, the tunnel excavation had reached Shaft No. 11, 1 175 ft. from Reservoir No. 4. The water standing in this shaft was drained away safely, but the flow from the loose material surrounding the shaft continued at the rate of 165 000 galls. per day for a number of days. Before reaching this shaft, the tunnel passed through a wall of light-colored clay, some 18 to 20 ft. in thickness, and so dense and compact that very slow progress could be made with the excavation. Only a small fragment could be removed on the point of a pick, and at times wedges were used to break out blocks of the material.

The finding of this wall of compact material to the eastward of this shaft was confirmatory of the diagnosis of the consulting engineer, for, apparently, this wall or dike of clay was holding back the water in the underground reservoir in exactly the same manner as was done by the body of clay found to the eastward of Shaft No. 1. In both instances the body of clay was so compact as to be practically impermeable to water.

The flow of water from the underground reservoir at Shaft No. 11 proved to be so strong that it interfered with the continuance of the tunnel work at that point, and, accordingly, the tunnel party was moved back to Shaft No. 1, from which point a branch tunnel was started in a northwest direction toward Shaft No. 8. This tunnel was begun in January, 1901, and on March 29th was completed to Shaft

No. 8, a distance of 427 ft. Only small quantities of water were found in this tunnel. A small quantity had accumulated in Shaft No. 8 and the short tunnel connected therewith.

Work on the main tunnel, at Shaft No. 11, was resumed in April, the water stored in the interstices of the rock having drained away by that time, so as to admit of the continuation of the tunnel excavation with some degree of comfort. The flow, however, continued for some time at the rate of from 50 000 to 60 000 galls. per day.

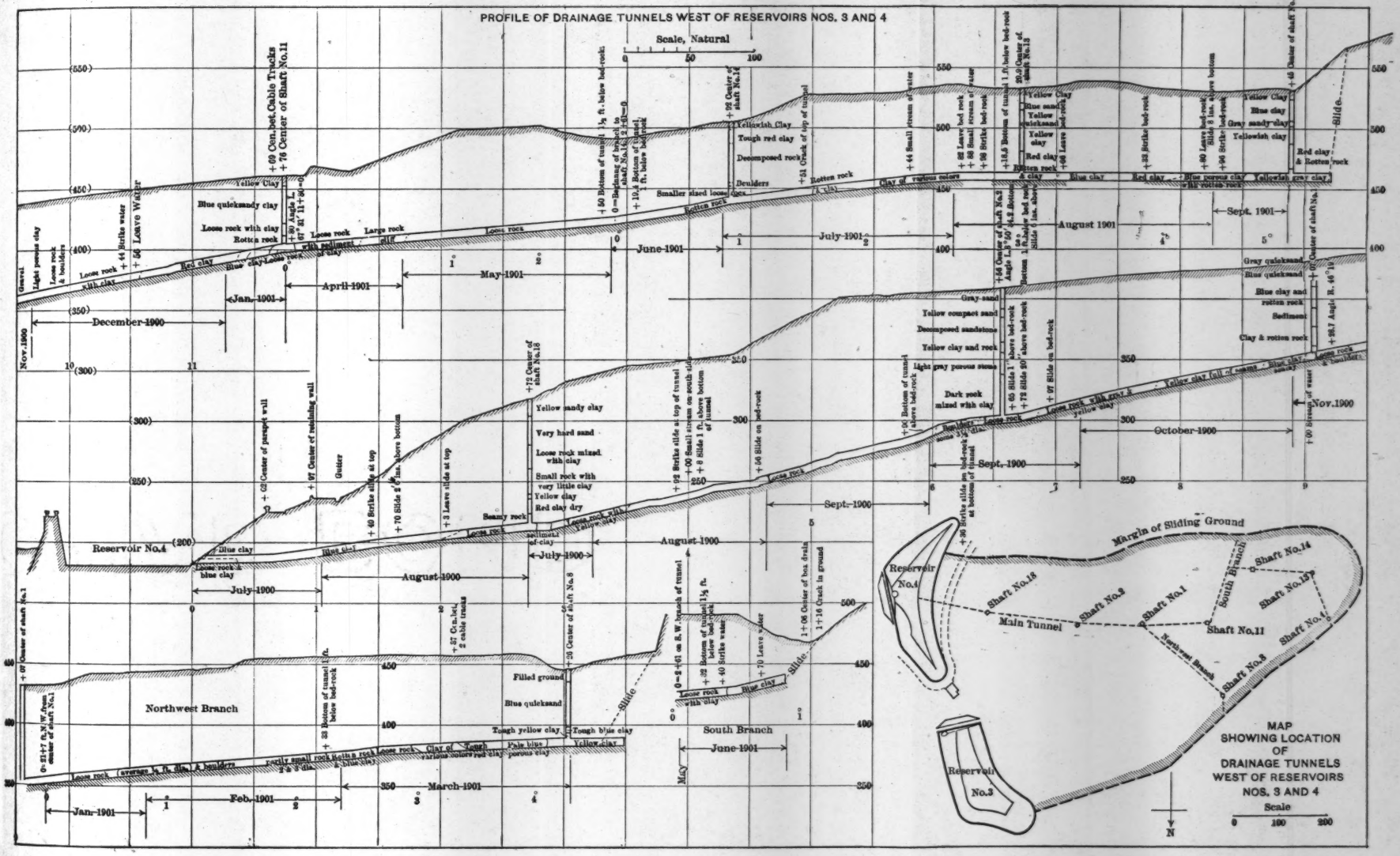
Instead of running west from this point, on the course laid down originally, it was decided to deflect the tunnel to the southwest so as to reach the southern limit of the slide, thus, with the northern branch, covering the entire width of the sliding ground and serving to cut off any and all bed-rock drainage from the west.

The southern rim of the slide was reached on June 14th, 1901, the tunnel excavation having passed beyond the limit of the water-bearing material and into dry clay. The line of the movement at the margin of the slide was defined plainly between two masses of yellow clay, the marginal slope being 1.2 vertical to 1 horizontal, from the tunnel level to the surface of the ground.

TABLE 9.—Cost of Drainage Tunnels, June, 1900, to December, 1901.—Expenditures for Materials and Labor, Exclusive of Surveys and Superintendence.

Date.	Linear feet constructed.	Average cost per linear foot.	Total cost.
1900.			
June	.....	.....	$87.71
July	157	$5.14	807.16
August	308	4.28	1 317.88
September	257	5.18	1 330.48
October	169	5.18	876.01
November	76	14.11	1 072.61
December	161	5.31	854.62
1901.			
January	131	7.54	987.77
February	167	4.66	777.80
March	180	5.18	932.29
April	96	9.35	898.26
May	175	5.46	956.17
June	170	5.72	972.48
July	182	4.78	870.78
August	217	5.01	1 086.80
September	61	5.30	323.00
October	.....	.....	9.32
November	.....	.....	.....
December	.....	.....	.....
Totals	2 507	$5.65	$14 161.14

PLATE XXVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIII, No. 984.
CLARKE ON A PHENOMENAL LAND SLIDE.



When the excavation had reached this point, the Committee decided to continue the tunnel to the extreme western end of the slide in order to insure the drainage of all large water deposits. This western extension of the tunnel was begun in June, 1901, and, passing through Shafts Nos. 14 and 13, was completed on September 11th, to Shaft No. 4 near the extreme western end of the sliding tract. The force was then disbanded. The total length of drainage tunnel completed during the fourteen months that the work was in progress was 2 507 lin. ft., costing, for material and labor, $14 161.

The wages paid were: Outside laborers, $2 per day of 10 hours; tunnel men, $2.25 and $3 per day.

The main characteristics of the material encountered in the tunnel are shown upon the profile of the tunnel line, Plate XXVI, but the following details may be of interest :

When the drainage tunnel was commenced, the initial point was established at the lowest point in the reservoir bottom, opposite the gate-house, from which point a 12-in. drain pipe, laid during the original construction, connected with the main sewer. Borings, made in 1895-96, had developed the fact that most of the reservoir bottom was either upon or within a few feet of bed-rock, while a few feet west of the bottom, and immediately underneath the west slope, was found a bed of clay which extended from 12 to 20 ft. below the reservoir bottom. Apparently, this depression in the bed-rock had been the ancient bed of the creek which had been filled by the slide, thus causing the creek to form a new bed further east and upon a higher level. It was expected that, for the first 75 to 100 ft., the tunnel would be in this clay bed and quite a distance above the bed-rock, and it was not known but that secondary lines of movement might be found as low as the bottom of the reservoir ; but this did not prove to be the case. For the first few feet a ridge of loose rock, 3 ft. high, was encountered, and after passing this loose rock dike only clay was found until the bed-rock was struck on its upward slope, at a distance of about 200 ft. from the mouth of the tunnel. For the first 70 ft. the tunnel grade rose 2 ft., and from that point the slope was uniform to the bottom of Shaft No. 18, where bed-rock had already been uncovered. While passing through this mass of clay no cracks which gave the slightest indication of any movement at the level of the tunnel were found.

At a distance of 164 ft. from the reservoir, the new tunnel passed under the short tunnel built in 1894 in the rear of the concrete retaining wall. It was in this old tunnel that an old tree stump or log of wood, in a fair state of preservation, was found, at an approximate depth of 40 ft. below the original surface of the ground.

The sewer, for the conveyance of the surface water from the watershed of King ravine into the Washington Street sewer, to which reference has already been made, consisted of 1 602 ft. of 24-in. terra cotta pipe and 148 ft. of 14-in. cast-iron pipe, a total length of 1 750 ft. Its construction was begun in July and completed early in November, 1901, and cost, for materials and labor, $5 123.

As has been stated, the drainage tunnels were completed in September, 1901, and since that date the only work in connection with the reservoirs has been to make frequent surveys of the lines established to determine the rate of the movement, and also to measure the flow of water from the tunnels.

The range lines established in 1895, when the limits of the slide had not been defined closely, were observed regularly until December, 1899. At that time it was decided to establish a new series of lines covering the ground more thoroughly than had been done heretofore. In pursuance of this plan range lines were marked out, crossing the sliding-land tract from north to south, at intervals of 100 ft. for the entire distance between the reservoirs and the apex of the slide. (See Fig. 6.) Points for observation were established along these lines at intervals of 50 ft., making a total of two hundred and sixty points known to stand upon ground which had been moving. These points were marked by a section of 1 or 1¼-in. iron pipe about 20 ins. long, driven flush with the surface of the ground, the top of the pipe being filled with lead in which a tack was driven. Surveys of these range lines have been made at frequent intervals since they were established, and the results of these surveys have been tabulated and plotted upon Plate XXV, showing the comparison between the rainfall and the movement of the slide.

In further explanation of this diagram it may be said that the monthly movement during the years 1895 to 1899, inclusive, has been computed from the average of readings taken at fourteen points established along seven of the original range lines, situated within the limits of a central belt, 200 ft. wide, extending from the north end of

DIAGRAM SHOWING RANGE LINES ESTABLISHED IN DECEMBER, 1899, COVERING THE ENTIRE AREA OF MOVING GROUND, AND OBSERVED SINCE THAT DATE.

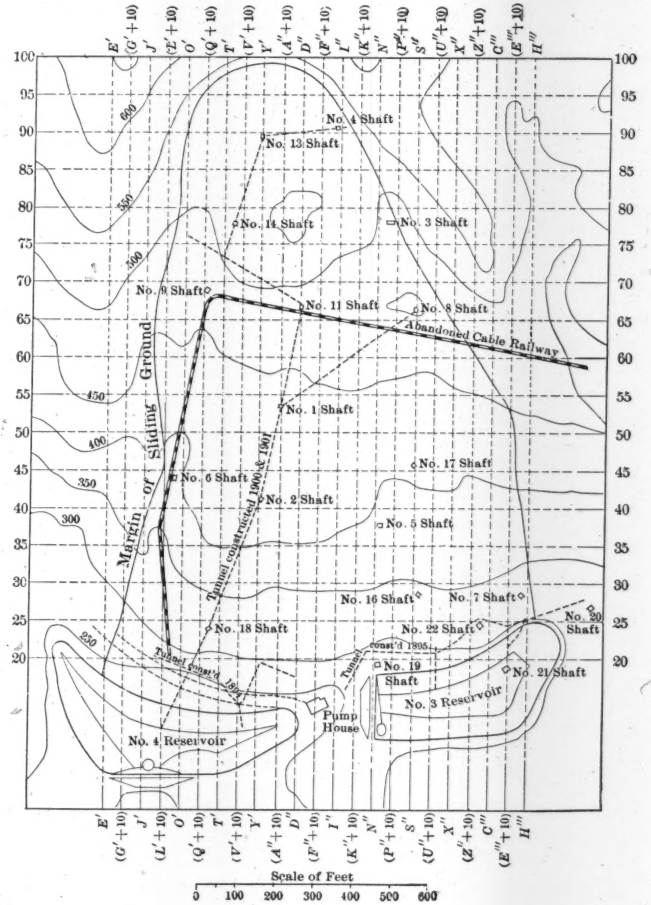


FIG. 6.

Reservoir No. 4 to the center of the "round top," west of Kingston Avenue and the old cable railway. The course of this central belt approximates closely the line of the maximum movement.

The points selected from the surveys made since December, 1899, have been taken from the central belt, and comprise fifty-one stations taken from seventeen of the north and south range lines. (The observations plotted show a movement somewhat in excess of the average of all the points, covering the entire area of the moving ground.)

From a study of Plate XXV and other data it is very apparent that the tunnels have been effective in draining away the water pockets and bringing about a condition of stability in the sliding mass. It is also to be noted how closely the decreasing movement of the slide seems to have corresponded with the progress of the tunnel work and the increasing area of drained ground.

The results of the drainage work thus far have been very satisfactory, for during the two years which have elapsed since the tunnels were completed there has been no appreciable movement of the slide. The surveys have shown some slight variations at individual points, from month to month, but these variations have not been confined to one locality and not always to consecutive points upon the same range line. Under the circumstances, it has been thought but fair to assume that the variations noted in the readings at various points have been due in part at least to errors in the instrumental work.

The instrument used in making these surveys since 1897 is a Buff and Berger transit-theodolite, No. 11, purchased expressly for this work. Care has been taken to insure the accuracy of the surveys which have been made from month to month, these having been repeated in whole or in part more than eighty times during the past eight years. As a rule, the readings have been taken from a single transit station at the end of each range line, a plumb-bob string, carefully supported, being used instead of a line rod. In order to be conservative in the matter, it has been considered that a variation of 0.02 ft. between consecutive monthly readings did not necessarily mean that a movement had taken place at any point during any particular month. If successive surveys showed an increasing variation from the original line it was then assumed that there had actually been a movement at that point. The photographs show that the surface of the slide is rough and broken, and hence it will

be seen that a sight at the tack head could not always be taken. It has therefore been assumed that for the purposes of the work 0.02 ft. was not an unreasonable allowance for instrumental error in the survey of the range lines.

The flow of water from the tunnel drains has been observed at frequent intervals since their completion, and the daily flow has been found to range between 10 000 and 15 000 galls. during the summer, and from 25 000 to 50 000 galls. during the winter. After severe rain storms the flow sometimes increases to 75 000 galls. per day for a short time.

The results achieved have been particularly gratifying, for, while the expense of the reclamation work thus far undertaken has been quite large, amounting to more than $31 000 during the years 1895 to 1901, exclusive of the outlay for engineering and superintendence, yet the results accomplished fully justify the course followed by the Water Committee.

The patience of the Water Committee in waiting for the plans of the engineers to be fully developed, and their wisdom and courage in executing these plans so thoroughly after they had been formulated, is especially worthy of mention.

The Committee can now look forward with confidence to the time, not far distant, when the reservoirs can be fully repaired and put into service, thus forming a beautiful and attractive feature of the park system, as well as fulfilling the useful purpose for which they were designed originally. They will thus save, for the use of the City, property which cost originally more than $500 000.

The work of replacing and repairing the broken linings of the reservoirs is yet to be done, and also that of making permanent the tunnel drains. It was at one time planned to begin this work during the year 1903, but, large expenditures being required for extending distribution mains, the reservoir work has been postponed.

Plans are in course of preparation for this repair work, and also for a concrete sewer to be built inside of the drainage tunnel, so as to provide a permanent outlet for all waters percolating through the body of the slide and reaching bed-rock. When this work is accomplished, the "land slide" at the Portland reservoirs will have become an event of the past, and will soon be forgotten by all but those who have been intimately connected with the work.

That the engineers who have been familiar with the details of this work have profited by their experience, is undoubtedly true, and it is with the thought that the lessons taught by the experiences related herein might be helpful to other members of the profession, that the writer has ventured to describe, with a fullness of statement which might not be justifiable under other circumstances, the difficulties encountered and overcome. If the record herein given proves to be helpful to any professional brother, then the purpose of the writer will have been accomplished.

The paper gives the history of the slide, down to the first of January, 1904, showing in detail the surveys upon which claims for the success of the reclamation work have been based. The writer believes that any claims he may have made are fully warranted by the results achieved, as shown by the charts submitted with the paper. That the result of a later survey, completed during the early part of March, 1904, portends an unexpected change in conditions formerly existing, may probably be taken as another proof of the "phenomenal" character of the movement.

As has been indicated, a resurvey of the range lines crossing the slide shows a slight change at a number of the points observed, a larger proportion of the points giving readings greater than for any former survey for two years. About 60% of the points observed gave readings ranging from 0.01 to 0.02 ft. in excess of former surveys. There were about half a dozen points which gave readings in excess of 0.02 ft., and only one as much as 0.05 ft., the average of points shown on the diagram, Plate XXII (east and west lines X', A" + 10 and D"), being 0.13 in. for the months of January and February, or 0.065 in. per month. This is indeed a small variation, and is significant only as an indication of the possibilities of the situation, which will require additional time to develop fully.

This unlooked-for change is unquestionably due to the unusual rainfall of the last four months, November, 1903, to February, 1904, inclusive. That this rainfall has been excessive is shown by the following figures compiled from the reports of the United States Weather Bureau in Portland.

The total rainfall for the months of November and December, 1903, and January and February, 1904, was 30.15 ins. This amount is 14.7% more than the average for these months for 34 years, and 27% more than the average for the same period during the past 21 years (1883 to 1904). It should be said, however, that the average for 13 years (1870 to 1883) was 30.4 ins., or slightly in excess of that for the present season.

The average rainfall during the months the Weather Bureau calls the "Wet Season"—September 1st to May 31st—is 42.39 ins., while the yearly average is 45.70 ins. Comparing the present season with the foregoing, the following may be noted.

Total precipitation November, 1903		10.71 ins.,	
" "	December, 1903	3.14 "	
" "	January, 1904	5.22 "	
" "	February, 1904	11.08 "	
			30.15 ins.,
" "	March 1st to 28th, 1904		8.64 "
Total			38.79 ins.,

the latter amount being about 85% of the yearly average.

The severity of these storms has also been shown by the volume of the drainage from the tunnels, which, upon a number of occasions, has been at the rate of more than 100 000 galls. per day. This shows in a striking manner the efficiency of the present tunnels. If this volume of water were not allowed to escape so freely, there would be trouble indeed, and the original "saw-tooth" character of the movement chart would soon reappear.

Considering the volume of rainfall and drainage, it is probable that the next survey will also show unfavorable results, for it will require some time for the ground to dry out after the rain has ceased.

That the movement is likely to continue for an indefinite period (if, indeed, it can be said that a general movement has already commenced), the writer does not consider at all probable. In order that the grounds for this confidence may appear, attention is called to Table 10, a comparative statement of the rate of the movement between January, 1895, and March, 1904.

TABLE 10.

Dates.	Length of period, in months.	Movement.		Remarks.
		Total, in inches.	Inches per month.	
January, 1895, to June, 1900	55	46.76	0.85	Reservoir linings wrecked during this period.
June, 1900, to November, 1901	17	1.68	0.10	Tunnel construction in progress.
November, 1901, to March, 1904	28	0.47	0.02	Period since completion of tunnels.

The last-named amount includes the movement just observed, which amounts to 0.13 in. for the months of January and February, 1904, and, with that amount included, the average recorded movement has been less than one-fortieth of the average before the tunnels were completed.

It may be noted that, by the last survey, there were very few points opposite Reservoir No. 3 which showed an increased reading, the greater number being along the axis of the slide opposite the north end of Reservoir No. 4.

The effects of the present severe storms are simply regarded as so many indications that the tunnels already constructed are not sufficient to drain thoroughly the entire mass of the sliding ground and its underlying bed-rock. The Water Board recognizes this condition of affairs, and, even now, has under consideration the construction of additional bed-rock drains, as branches of the main tunnel; and also a system of sub surface tile drains to cover certain portions of the surface, provided later surveys afford conclusive proof of the necessity for this additional drainage work.

The result of the changes, already noted, will be to delay somewhat the work of reservoir repairs, but there is no doubt felt of the ultimate success of the reclamation work and the final restoration of the reservoirs.

* * * * * * & *

During May, 1899, while the writer was engaged upon the investigation of the slide at the Portland reservoirs, there appeared in the city papers brief accounts of an immense land slide which had just occurred near Cape Meares, about 70 miles west from Portland.

The first descriptions of the slide, its size and the rapidity of its movement (width ¼ mile, length 4 miles, and movement 2 ins. per hour) were sufficiently startling to awaken a desire to examine the ground and see if the conditions actually existing there were in any way comparable with those which had been studied so long at Portland. A few weeks later, therefore, the writer undertook the journey, partly by rail and boat, but chiefly by stage or mudwagon, crossing the Coast Range, and occupying 36 hours or more, before reaching the slide. The time spent in the vicinity was three and a half days, and the greater part of two days was occupied in mapping the slide and noting its characteristics and progress. From the notes then taken the map, Fig. 7, and description of the slide have been prepared.

The slide is in a shallow depression lying immediately north of the range of hills of which Cape Meares is the western extremity, and is about 5 miles south of the entrance to Tillamook Bay, Oregon. The upper or eastern portion of the slide is covered with a dense forest, and it is not easy to determine the exact limits of the moving ground, but enough is known to warrant the statement that the length of

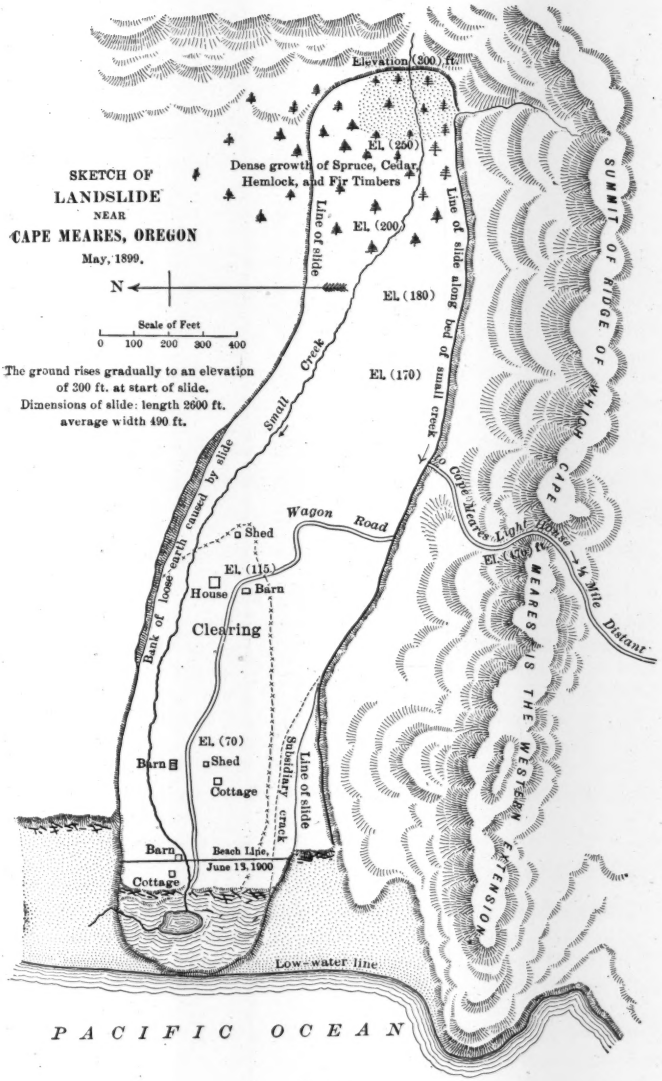

SKETCH OF
LANDSLIDE
NEAR
CAPE MEARES, OREGON
May, 1899.
N
Scale of Feet
0 100 200 300 400
The ground rises gradually to an elevation of 300 ft. at start of slide.
Dimensions of slide: length 2600 ft. average width 490 ft.
Elevation (300) ft.
El. (250)
Dense growth of Spruce, Cedar, Hemlock, and Fir Timbers
El. (200)
El. (180)
El. (170)
Line of slide
Line of slide along bed of small creek
Small Creek
Bank of loose earth caused by slide
Wagon Road
Shed
El. (115)
House
Barn
Clearing
El. (70)
Barn
Shed
Cottage
Subsidiary crack
Line of slide
Barn
Beach Line, June 13, 1900
Cottage
Low-water line
PACIFIC OCEAN
SUMMIT OF RIDGE OF WHICH CAPE MEARES IS THE WESTERN EXTENSION
Cape Meares Light House
El. (470) ft.
½ Mile Distant

FIG. 7.

the slide from the ocean beach to its eastern extremity is a little less than ½ mile, and the average width about 500 ft. The area of the moving ground is approximately 30 acres.

The larger part of the moving ground is covered with a dense forest of fir, spruce and smaller growths, and little can be seen which indicates in any way the probable depth of the movement. The movement has been sufficient to cause many trees to fall around the margin of the slide, and many more will soon follow.

The head of the slide reaches an elevation of about 300 ft. above the ocean beach in a distance of about ½ mile.

From present appearances, and from information derived from those familiar with the ground as it was before the slide occurred, it can be said that the slide had its origin in a small tract of swampy ground, perhaps 200 x 300 ft. in extent, through which ran two small streams of water, each perhaps 12 ins. wide and 1 or 2 ins. deep. One of these small streams finds its way to the beach along the base of the Cape Meares ridge, the southern limit of the slide being for the most part along or within a few feet of the bed of this stream.

The second small stream crosses the upper end of the slide to near its northern border, and thence finds its way to the beach, its course being almost, if not altogether, within the limits of the moving ground.

On the south, the line of the movement is defined very clearly, as it follows, for the most part, the bed of the creek until within about 400 ft. of the beach, where it leaves the channel of the stream and cuts through higher ground on a more direct course. Along the northern border, the ground is broken more irregularly; piled in ridges in places, with crevices between; but none of the latter was of any considerable depth.

At the head of the slide the horizontal movement has been sufficient to remove all timber and surface earth from a section of the hillside about 400 ft. long, from north to south, and from 50 to 70 ft. wide, measured along the slope of the bank. The slope of the clay bank, where exposed by the slide, is from 25° to 40°, and the vertical movement at that point amounts to about 30 ft. No chasm was to be seen, indicating a deep movement at that point; the ground had simply settled away from the adjoining bank, and all crevices had been filled with the surface earth loosened by the slide. At one point a rocky ledge had been exposed, and all around the upper margin of the slide the upturned trees indicated but a slight depth of surface earth.

While the upper or eastern portion of the slide is hidden from view by a dense forest, the lower part has been cleared of timber to a great extent, and is partly under cultivation; but, at the beach, existing conditions bear the strongest testimony to the force and magnitude of the movement. The height of the bank above extreme high

tide varies from zero at the center of the slide to 10 ft. above at the north side, and 20 to 25 ft. at the southern edge of the movement. Excepting at either margin of the movement, the portion of the bank above high tide shows no appearance of having been disturbed, although it has been moved 200 ft. or more from its original position, but the beach in front, for another 200 ft. or more, has been thrown up into ridges or folds by the immense pressure from the slide.

This movement has extended out to low-water line or beyond, and a nearly vertical bank, 10 ft. in height, formed of clay and sand in large masses, presents a bold front to the action of the waves, which slowly wear it away.

As far as known, the movement began on or about May 10th, 1899. By measurement, it was found that the total movement from May 10th to June 10th was 228 ft., or an average of 7.36 ft. per day.

From June 10th to June 12th it was at the rate of 4.01 ft. per day.

From June 12th to June 13th it was at the rate of 3.26 ft. per day.

From the foregoing, it appears that a falling off in the rate of the movement had already commenced at the time the examination was in progress. Considering the not excessive slope of the hillside, it was rather surprising to find that the movement had continued for such a length of time, and it did not appear unreasonable to suppose that it would soon cease altogether. This, however, was merely conjecture, for the movement no doubt depended upon the quantity of water which found its way under the slide, near its head, and what its volume might be no one could tell.

Some months after the writer's return from this examination he was informed that an entire cessation of the movement had taken place which must have occurred very soon after the last measurement was taken, June 13th, 1899.

The writer has no report regarding the condition of this slide later than June 13th, 1900. There had then been no resumption of the movement during the winter which intervened since the measurements were taken. It was also reported that the front of the slide had been worn away by the action of the waves as far back as the dotted line in Fig. 7.

DISCUSSION.

Mr. Dillman. GEORGE L. DILLMAN, M. AM. SOC. C. E. (by letter).—The writer's acquaintance with this ground began in 1891, when he had charge of the construction of the cable road (shown in Plates XVII, XVIII, XIX and XXII) for the contractors. In 1898 he was employed by the City of Portland to study and make a preliminary report on the sliding-land case, then in litigation. In 1899 he was one of the witnesses for the city at the trial mentioned.

Months before the completion of the cable road, it was known that the Cable Company would default in payments. The writer's business was to save as much as possible from the wreck, which may account for his neglect to appreciate the importance of several facts developed during construction. At any rate, their significance was not appreciated until later. These facts, as memory serves, include the following:

1.—From Boring No. 3 to Shaft No. 9 (see Plate XXII) the road was in a cut. Shortly after the grading was done, cracks in the slopes indicated motion, thought at the time to be local. These were obliterated by sloughing, after the rails were laid, at one time covering the finished track several feet deep.

2.—The track mentioned had a tendency to buckle, and both the slot and the running rails were cut to relieve the pressure. This part, however, was on a heavy grade (about 35%). The plan called for very light construction, so that insufficient anchorage might have been considered the cause of this trouble.

3.—The level track northward from Shaft No. 9 got badly out of line, the derangement not being confined to fills, but extending into cuts. The derangement on fills was thought to be from shrinkage; in cuts it was thought to be from the same local movement noted in Fact 1.

4.—In re-aligning the track northward from Shaft No. 9, there was considerable friction between the foremen and the engineers, the former complaining that the engineers could never run their lines twice alike, the latter that their stakes were disturbed.

5.—Expert timber men made failures in felling trees in the direction desired.

6.—There was a swampy hillside north of the timber-trestle bridge across what was later Reservoir No. 4.

7.—At one time the bents of this timber trestle were very much out of plumb, and their tops were jacked back, down hill.

In the examination of 1898, the first pertinent fact of importance developed was the lack of coincidence of the surface of movement and

the depths of excavation, being many feet below Reservoir No. 3 and many feet above Reservoir No. 4. Mr. Dillman.

There was a rupture of the lining of Reservoir No. 4 at about the bottom of the reservoir, which is, however, best accounted for by its construction. The clay back of the facing (concrete and asphalt) was carefully puddled. The water was in the reservoir several days, then quickly drawn off, so that any small seepage through the facing, or backed-up ground-water, would act with its full head on this weakest point. Thus, the reservoir lining contained within itself all that was necessary to its own destruction under the conditions imposed. The rupture near the bottom of Reservoir No. 4, therefore, had no necessary connection with the slide.

The extent and amount of the movement had been ascertained at this time. Colonel Mendell had directed investigations which were exhaustive and conclusive. His conclusions have been verified by the subsequent work of drainage, as shown by the paper.

The investigation of this slide is by far the most complete of which the writer has any knowledge. It seems to make plain that the water does the damage, not so much by its volume or weight as its pressure, and, at the surface of motion, acts like a myriad of jack-screws to lift, and at the same time lubricate it, the same pressure forcing the water into and through otherwise impervious strata.

The writer knows of many slides which have been cured by drainage, and knows of no failures to do so when the remedy was applied properly, and in the right places.

There are several sermons in this slide, one of which is on the fallacy of taking generally accepted facts for verities. Prior to this law suit, everybody seemed to know that the excavation of the reservoirs caused the slide. The Northwest is full of slides. The construction of every railroad develops them. The fact that cutting off the toe of a slope with sliding tendencies often starts a slip was warrant for this general belief. The traces of the slide on the surface were pretty well hidden by undergrowth. There were no buildings. The cable road was abandoned, but this might have been from other reasons than failure to maintain it. Engineers, looking casually at the reservoir failures, granted it. A prominent and capable engineer, well acquainted with slides, refused to investigate it for the city because he did not care to be bullyragged by contending attorneys over a point, to him, so evident. The predecessor to the City Attorney who won the case had made some investigation, and told the writer just prior to the trial that the only ground for the city to stand on was to deny liability first, then get the damages as low as possible; that there was no use in contending against such obvious facts.

Mr. Dillman. The investigations, under the direction of Colonel Mendell, indicated that there might be some doubt. The City Attorney, following this lead, found evidence of motion prior to the excavation of the reservoirs. Then the cable road operation and difficulties during construction seemed pertinent. Officers of the militia had had their target ranges on this ground, and noticed the derangement of stakes which should have been in line. Men were found who, when boys, had herded cattle on this ground and had lost cattle in the cracks at the head of the slide. All this was made so plain to the jury that they were absent only long enough to frame a verdict for the city. After the trial, everybody knew that this was an ancient slide. It was astonishing to find that so many people had known it all the time.

Mr. Adams. ARTHUR L. ADAMS, M. AM. SOC. C. E. (by letter).—It seems very probable that no land slide has ever received such exhaustive study as has been given to the one described in this paper.

Few slides have been productive of more embarrassing results, and engineers in general will unite in the hope that, as a reward of such long and patient effort as that described, this one has now "become an event of the past."

The recounting of methods by which great difficulties are finally overcome is always of value, and to this the paper is not only no exception, but, on the contrary, it ranks, in the estimation of the writer, among the best of its class in engineering literature.

Land slides are by no means uncommon on the Pacific Coast, from San Francisco north. The mountainous character of the country, the clayey nature of the soil overlying the bed-rock, and the heavy seasonal rainfall, are all favorable for their production. Scarcely a winter passes in which railroads are not repeatedly blocked, even on lines which have been in operation for years, and if the rain chances to be unusually abundant, or unusually protracted at any time, the recurrence of such slides becomes correspondingly frequent.

In many localities the slopes, in their natural state, are lying at what is virtually their maximum angle of repose in wet weather, and what may seem to be an insignificant side-hill cutting may be sufficient to destroy the existing equilibrium and start motion. This is also true of many large slides of broken trap rock, along the slopes of which railroad lines, in places, are maintained with the greatest difficulty. Even the wash of banks, incident to unusually high water on the Columbia River, for instance, has been sufficient to set in motion land on which improvements had existed for years without suspicion of instability.

During the building of the Portland reservoirs, the writer was in charge of the construction of a system of water-works at Astoria, Oregon, where the geological conditions were almost identical with

those about Portland, and where slides and land of doubtful stability were so abundant that great caution was called for in locating reservoirs and other permanent structures, and made it necessary to use slip-joints and other special devices in the pipe system where unstable ground could not be avoided. He was therefore a frequent visitor, and a much interested observer of the progress of events, at Portland. Mr. Adams.

The author expresses the hope that some of the engineers engaged by the plaintiff, in the trial of the interesting case of the King Real Estate Company *versus* The City of Portland, will avail themselves of the opportunity for discussion afforded by the publication of his paper. This the writer does more willingly because the ruling of the judge, in the trial of the case, prevented any review, by engineers for the plaintiff, of the expert evidence of the defense, with the result that the very essence of difference in expert opinion was obscured and lost because it was never driven home to the minds of the highly agricultural jury, carefully selected in compliance with law from without the limits of Portland, to try this very technical case. The experts engaged by the plaintiff were called in too late to make any suitable presentation by way of exhibits, an indispensable prerequisite to success, especially before a jury; while the defense, as the result of a year's preparation, combatted with a magnificent line of exhibits, of which the engineer-authors had just reason to be proud. The jury was duly overwhelmed; the engineers engaged by the plaintiff were not permitted to review; with the result, as has been stated, that the essential point of difference in the expert opinion was obscured and lost on the jury, to the satisfaction of the defendant and the grief of the plaintiff.

There was substantially no difference of opinion:

1.—Concerning the existence, character and dimensions of the slide, as it existed at the time of the trial;

2.—That without water there would, of course, have been no movement, whether or not the reservoirs had been built;

3.—That numerous instances of local surface sliding had been observed on the land prior to, as well as during, the reservoir construction;

4.—That there had been trouble, prior to the commencement of work on the reservoirs, in maintaining the surface and alignment of the cable road;

5.—That no deep-seated or general movement of the greater slide had ever been discovered or credibly suspected prior to excavating the reservoirs.

The fundamental question really involved in the expert evidence did not concern these matters at all, but was:

Had the great slide been in motion during a period of years prior to excavating the reservoirs? If not, was this excavation the disturb-

Mr. Adams. ing cause which upset the pre-existing condition of equilibrium and started or renewed the motion?

It will be observed in the statement that whether or not this land had at an ancient time been in motion is not involved in the technical controversy, no matter what bearing it might legally have upon the question of damages; but only the question as to whether it was permanently at rest prior to commencing work on the reservoirs.

There are several important matters, in addition to those mentioned in the paper, which are necessary for a full understanding of the situation. They are as follows:

First.—The high ridge lying immediately east of the two reservoirs and across the front of the slide, and through which the ravine in which the reservoirs were built passes at the narrow outlet closed by the dam of Reservoir No. 4, is a rocky barrier of unquestioned stability. (See Plate XVII.)

Second.—The small streams originally flowing during the rainy season in the depressions in which the reservoirs were afterward built were not of an erosive character.

Third.—In excavating the reservoirs, not only were the slopes cut back and increased in pitch far beyond the point of stability for wet clay, but the ravines were much deepened by making the reservoir bottoms much below the prior existing natural elevations. The paper does not give the various depths of this cutting, but they were shown as a part of the defendant's exhibits. On page 327 the author mentions a 41-ft. cutting, but the writer's recollection is that it was generally much less.

Fourth.—The tract, including a large part of the land later found to be in motion, had been subdivided and staked out, as indicated in Plate XVII, five years before the cable road was built and six years before the excavation of the reservoirs was started; and it was proven by Mr. J. A. McQuinn, the engineer who had made the subdivision and who had thereafter resurveyed the tract, closing on the original stakes, that there was a complete absence of any general movement between 1887 and 1893.

The theories and interpretation of evidence on the part of the defense tended toward the demonstration of the existence of the large general slide, as described in the paper, presumably dating in origin far into the past, and which, up to the present, had not come to a condition of repose; in short, that the ground had been moving every year, but without this movement having been discovered previously.

In the opinion of the writer and others associated with him, there were many insurmountable difficulties in the way of the acceptance of this theory; while the hypothesis that the slide, even though of ancient origin, had long ago come to a state of rest against the rocky barrier

directly in its path, to be again set in motion by the removal of the supporting earth and broken rock, satisfied, in their judgment, every material point of evidence as to fact, and was fully warranted from a study of the physical conditions. Mr. Adams.

Some of the difficulties in the way of the acceptance of the theory of the defense were:

First.—The failure of the Portland Water Committee and its engineers, during two years of study and investigation prior to commencing work, to discover this movement, although cautioned by intense public opposition to the sites proposed, because of their feared instability.

Second.—It was utterly at variance with the evidence of Mr. McQuinn, who had subdivided the tract and resurveyed it years later, as hereinbefore recited.

It was also at variance with much other, less conclusive, but still good indicative, evidence that there had been no movement for a much longer period than that covered by Mr. McQuinn's observations, and none at any prior time, as far as known.

To substantiate the claims of continuous movement, it was necessary to attribute the various difficulties with the cable line, during the short period of its operation, from May, 1892, until the following September, to this cause. Therefore, in the trial, great stress was laid upon this by the defense.

If the phenomena observed on this road are accounted for by the existence of local surface slides and the exigencies of operating a new road in this locality, a cable road at that, during the first year of its completion, then the theory of the engineers for the plaintiff is incontrovertible and all material evidence is reconciled. The writer is convinced that, not only can these phenomena be thus accounted for, but that, on the contrary, the theory of a general, deep-seated movement cannot account for them at all.

As to the character of this cable road, as given in evidence: It was built in the rainy season. On Kingston Avenue, in the vicinity where the greatest lateral movement was observed, the track was on a fill, 30 ft. high, built on a hillside so steep that the bank was 20 ft. higher on one side than on the other. This fill had been built just before track laying, and before the heaviest rains of the season had set in. On the east and west line, the grade was largely in quite deep cutting, and the evidence showed that the roadbed was so soft and yielding that during construction it was constantly forced upward immediately adjoining the track, doubtless from the weight of the earth in the adjacent banks. The trestle, which crossed the ravine where Reservoir No. 4 was afterward built, carried the track on a 20% grade, and was built in contact with a high end-bank, which, under the action of heavy rains, was amply sufficient to cause the trestle supports to be pushed seriously out of plumb.

Mr. Adams. The evidence showed that on Kingston Avenue, before referred to, the lateral movement of the track was from 16 to 20 ins. during a period of from 4 to 5 months, a rate of movement greatly in excess of anything which has at any time been observed in the big slide.

It was testified by the trackmen that the 90° curve on Kingston Avenue, in spite of its being at or near the zone of maximum movement of the large slide, had moved very little, if at all.

Again, if the same 90° curve had been moving, with the general slide, 1 or 2 ft. a year, with motion induced by the push of a saturated mass in the upper end of the slide, such an action would have resulted in the uniform crowding and buckling of the track to the east, as far as the point marked "Road to Base-Ball Ground" (Plate XVII). On the contrary, the evidence showed the track to have been pulled apart and the yokes broken on this section, a phenomenon readily produced by the lengthening of the track from unequal settlement in a very soft roadbed.

Farther to the east, there was buckling of the track, according to the evidence, but it was too irregular in character, in places of occurrence, and in total amount, to harmonize in any way with the hypothesis of a general movement such as has been shown to exist since the reservoirs were excavated.

On page 332, in the extract from the report of the engineers for the Water Committee, the existence of surface slides is mentioned. The evidence in the case was full of observations of such local slides. A barn, once on the land, had been moved out of position by this cause. In one place the trees were observed to be leaning to the east, and there were various other manifestations, none of which could be reasonably attributed to the action of a slide from 60 to 90 ft. thick, moving from 1 to 2 ft. a year.

The writer has said that a study of the physical conditions, alone, also warrants the hypothesis which reconciles so effectually all the evidence of fact.

Attention is called to the exceeding slowness of the movement, as indicative of how little equilibrium was disturbed.

It must be admitted that this slide, in its movement eastward, granting that it was formerly in motion, when sufficiently in contact with the rock barrier, extending entirely across its front save for the gap closed by the lower dam, must come to a state of rest. It is not conceivable that, to reach this state of rest, a slide, consisting mainly of rock and stiff clay which without support moved only 1 ft. a year, would need to rise against the barrier to the full height of its natural surface and thus wholly close the natural ravine there existing. On the contrary, it might reasonably be expected to rest when in but partial contact, and the extent of this contact necessary to continued repose might be, and probably was, relatively very

small. When dealing with forces not far from the point of exact equilibrium, grave consequences may easily follow seemingly insignificant causes. Mr. Adams.

The great weight of the moving mass has been urged in proof of the irresistible character of its movement. Mere weight means nothing in determining its energy unless its speed also is considered. If the slide were only 3 ins. thick and moving as slowly as 4½ ins. an hour, it would develop as much energy as when 75 ft. thick and moving 12 ins. a year.

With a slide claimed to be of geologic origin already in contact with a barrier which admittedly must stop its progress; with instrumental surveys having been made over its surface for six years; and exhaustive borings and examinations made by careful engineers charged to determine its stability; and with no movement discovered until excavations across almost the entire width of its front had been carried, in most places, below its plane of movement; with the discovery, at that time, of a rate of motion of something like 12 ins. a year, followed by the rapid opening of large cracks (Page 342) around the boundaries of the slide; and the tracing of these lines of fracture on both sides to points of contact with the reservoirs almost exactly coincident with the extent of their excavations, modified slightly on the south by the topography (Plate XVII), the writer is impelled to the seemingly inevitable conclusion that, whatever the geological history of this land may have been, this slide was in a quiescent state prior to its being set in motion by the excavation of the reservoirs.

JAMES D. SCHUYLER, M. AM. SOC. C. E. (by letter).—An opportunity to add a few words to the discussion of this admirable paper is eagerly embraced, although protracted absence from home on professional work, out of touch with the Society literature, had almost prevented the writer from participating in a subject of deepest personal interest to him. Mr. Schuyler.

During 1894, the writer, with the title of "Consulting Engineer," was employed to design and construct the two reservoirs, Nos. 3 and 4, involved in this landslide, as well as two others, Nos. 1 and 2, on the east side of the river.

The late Isaac W. Smith, M. Am. Soc. C. E., was the Chief Engineer of the Water-Works, and Mr. Clarke was his Principal Assistant, in charge of the construction of the main pipe line from Bull Run to Portland.

The construction of the four reservoirs, in the few short months of fair weather available in the moist Oregon climate, was a most herculean task, and, under ordinary conditions, should have occupied at least two seasons, but, as the realization of a pure water supply came nearer, there was great public clamor for the completion of the works

Mr. Schuyler. before the close of 1894, and as the contractors for the pipe making and laying needed to be stimulated to extra exertion by the evidence of preparedness on the part of the city's engineer to receive water through the pipe; and, furthermore, as a large money saving, amounting to $100 000 or more, was to be made by eliminating the cost of pumping the city's water supply by an early completion of the system, there was a strong incentive for pushing work on the reservoirs as rapidly as possible.

The writer's judgment inclined to more deliberate action, but the public eagerness to enjoy Bull Run water at the earliest possible moment, and the possible saving in cost of operation seemed sufficient inducement to overcome his conservatism in this regard.

Delay in completing the reservoir until 1895 would have given the slide opportunity to develop without destroying the linings, but would probably have led to the abandonment of the reservoir sites. This would have been a misfortune, for the reason that these reservoirs are in the most convenient locality possible, and they can yet be made available for reservoir purposes by a thorough drainage of the slide. Any substitution of them, by the construction of reservoirs of equal capacity at corresponding elevations elsewhere, would have been unquestionably more expensive to the city than the losses involved in interest on the cost of the unused portion of the work and the cost of that part of the reservoir linings which has been shattered and destroyed by the movement; added to the subsequent expenditure made in exploring and draining the slide. Events have proven that the city has really been a gainer by what has occurred, rather than otherwise.

It may be considered as particularly fortunate that the gate-houses and pipe connections were designed in such a careful manner that the breaks in the reservoirs had in no wise interrupted the service, and the works are as efficient as though the reservoirs had never been injured by the slide. Even the pumping plant, operated by the water falling from Reservoir No. 3 to Reservoir No. 4, was not stopped a day in its operation of the high-level pumps.

The developments, described so minutely and interestingly in the paper, of the extent, depth, area and volume of this remarkable slide, the very slow rate of its movement, and the effect produced upon it by drainage, constitute an impressive array of facts teaching a lesson of the importance of far-reaching investigation prior to beginning important construction. The preliminary investigations and borings, referred to in the paper, which were made prior to the engagement of the writer, were reviewed carefully by him at the beginning of the work, and appeared to be conclusive and satisfactory in demonstrating that the foundations of the reservoirs were entirely stable, and that the numerous local land slips, appearing on all sides of the reservoir sites, were not deep-seated, and would be cut out by the excavations planned.

Mr. Schuyler.

The writer examined carefully all the hill slopes above the reservoir many times during the excavation of the basins, and never saw the slightest indication of the general slide such as developed subsequently. As stated, the small surface slips were numerous, not only on the west side of both reservoirs, but on their east, north and south slopes. They were caused evidently by super-saturation of the soil, standing on slopes steeper than the natural angle of repose, and the manifest remedy was drainage. To accomplish this drainage in the most efficient way possible so that the linings might not be disturbed when the reservoirs were finally in service, an elaborate system of slope and bottom drainage was planned and carried out, consisting in part of numerous drive-well points driven into the slope, to relieve the water as far back of the surface as possible. These were connected with tile drains laid up and down the slope at frequent intervals, in trenches filled with broken stone. These tile drains discharged into sewer pipes laid at the toe of the slope, underneath the lining, all around the reservoir, emptying into the city sewers outside the dam.

The most extensive and troublesome of these surface slips was the one which constantly discharged mud into the excavations of Reservoir No. 4, about on the line of the cable road. This slide extended west almost as far as the curve, the distortion of which is shown in Fig. 2, Plate XIX, and was lubricated constantly by spring water following down the cable road track. The writer noticed the distorted rails at this curve on several occasions during the season, and ascribed their condition to the local settlement, and the slide below it.

From all the phenomena observed by the writer during the construction of the reservoirs, he is of the opinion that the large, main slide, the outlines of which were revealed in the year following the excavations of the reservoirs, was in a state of rest throughout the year 1894, up to the time when the cracks first appeared in the west slope of Reservoir No. 4, in August. The protrusion of the upper layer of clay in a marked line, and at a rapid rate, out beyond the face of the excavation was sudden and startling, and although at the time it appeared to be but a deeper manifestation of the old troublesome cable road slip than had yet been apparent, it is now clear that it was the beginning of the movement of the big, main slide. That this was caused by the removal of the toe of the slide in the excavation of the reservoirs is indicated chiefly by the fact that it occurred in the dry season and at the latter end of the summer. Had the slide been moving during all the year previous, the manifestation of it could not have been overlooked. Evidently, it was in a state of unstable equilibrium requiring but small cause to start moving again, and, although the volume of excavation removed from the reservoirs was but 3% of that of the moving area, as pointed out by the author, it was the key which held the mass from continuing its ancient movement.

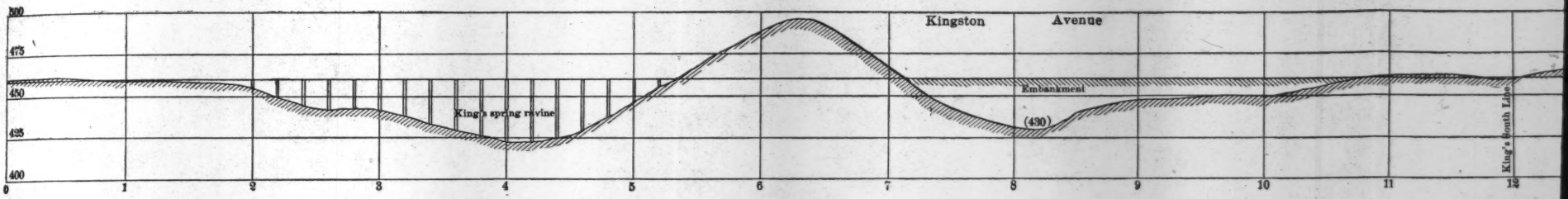
Mr. Schuyler.

All other great slides of this region have their greatest movement during the rainy season, while this one apparently started in August, the driest part of the year, with a movement of ½ in. per day for several days, although the subsequent maximum rate was but 12 in. per annum.

During the progress of this work, in the early part of 1894, a land slide occurred at Portland Heights, about half a mile southeast of Reservoir No. 4, which suddenly overwhelmed a valuable dwelling with mud, and destroyed its contents. The volume of earth which slid was probably less than 5 000 cu. yd., originating in a street embankment on the heights near the edge of a high bluff. It descended a slope steeper than 1 on 1, a vertical height of from 300 to 400 ft., and, after starting, its movement was rather in the nature of an avalanche of mud than that of an ordinary land slide. It was evidently something of this character which was expected by the Committee of 100, referred to in the paper, when they protested against the location of the reservoirs in the City Park, as the Portland Heights slide was of very recent occurrence and fresh in the mind of everyone when they made their protest. Such a torrential movement alone could have displaced the water in the reservoirs and caused a sudden flood, and it was against an accident of this type that the Committee expressed their fears. The Engineers of the city saw no evidence of the possibility of a slide of that character in the vicinity, and so favored the continuance of work. They were at least as well able to judge as the Committee of 100 novices and laymen, and they certainly did not see a sign of the deep-seated ancient slide, the slow movement of which afterward put the reservoirs out of service.

That this slide was caused primarily by water and the lubrication which water afforded to the under surface of the moving mass, is so manifest as to be axiomatic. Practically all land slides have the same moving cause, and would remain stable but for water.

The most notable and extensive land slides which have come under the writer's observation occurred on the line of the Canadian Pacific Railway, a few years ago, as the result of irrigating high bench-lands along the Fraser River. The litigation and investigation which followed was described by the writer in a paper read before the National Irrigation Congress of 1895, held at Phœnix, Arizona. The cause of these slides was similar to those which developed the slide under discussion, *viz.*, removal of the toe of the old slide, in a state of rest, by a railway cut, and the continued application of water for irrigation, in the same saturating excess which produced the slide before the railway was built. Irrigation caused the slide; after the original movement it reached a state of equilibrium where it was no longer moving, during which period the railway location was made; subsequently, the railway cutting removed the toe of the slide and started



PROFILE OF JEFFERSON STREET CABLE RAILROAD AND EXTENSION FROM 15TH STREET WEST

(December 1891)

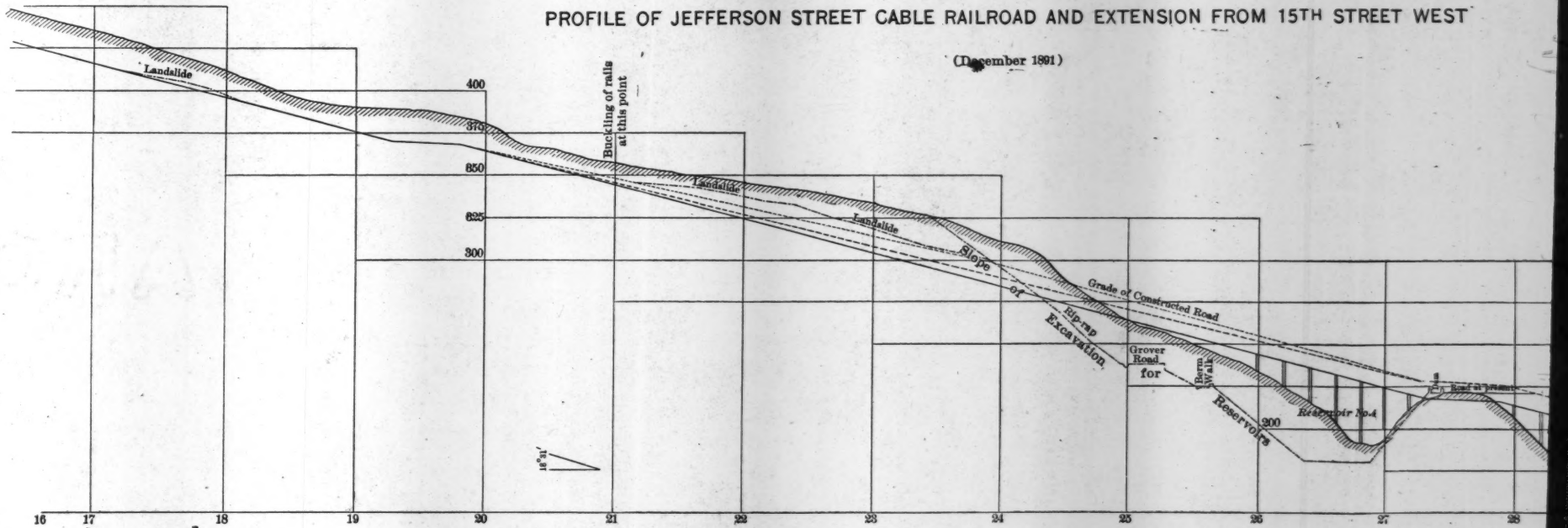
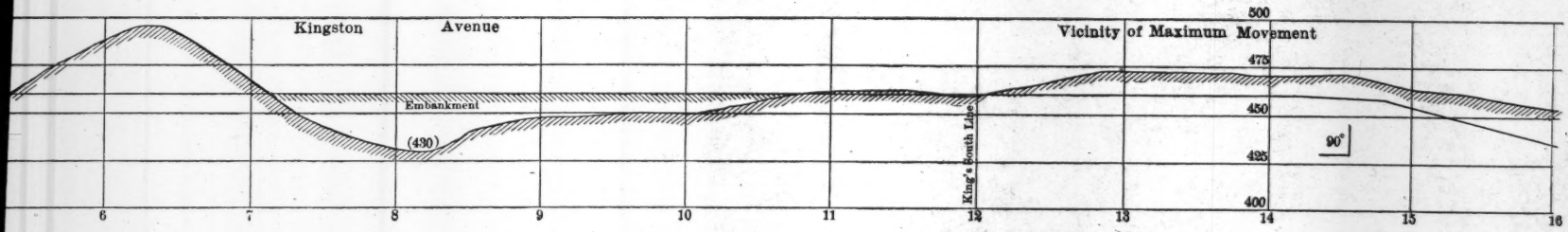
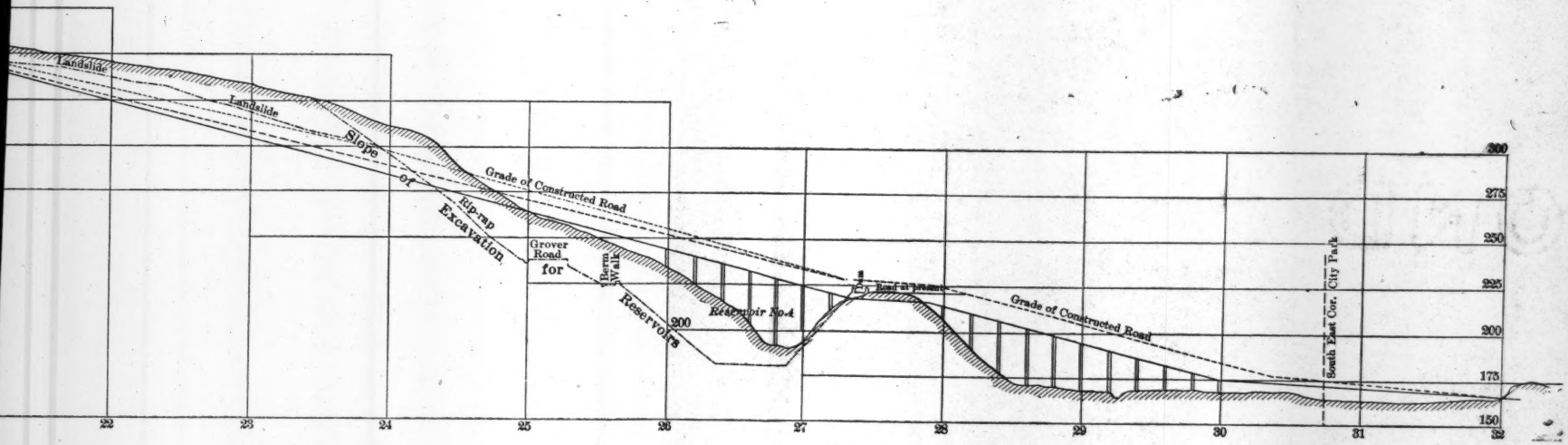


PLATE XXVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIII, No. 984.
CLARKE ON
A PHENOMENAL LAND SLIDE.



F JEFFERSON STREET CABLE RAILROAD AND EXTENSION FROM 15TH STREET WEST

(December 1891)



its movement afresh, and continued irrigation kept the ground lubri- Mr. Schuyler.
cated and in motion until the courts enjoined the irrigation, and the trouble ceased.

D. D. CLARKE, M. AM. SOC. C. E. (by letter).—In reviewing several Mr. Clarke.
matters referred to by Mr. Adams, the writer desires to call attention to Plate XXVII, which shows the original profile of the cable railway from the head of Jefferson Street to the terminus of the line beyond the northern limit of the slide. From this profile it appears that the road was built several feet higher than the grade originally established, probably in order to lighten the cutting west of the reservoir crossing. The "landslides" in this cut were caused by the caving of the sides of the excavation during the early stages of the work.

Upon this profile has been platted a cross-section of the excavation for Reservoir No. 4. It was near this crossing that a cut of 40 ft. occurred. It may also be stated that this cross-section shows practically the worst point to be found along the full length of both reservoirs, and it can be taken as an extreme example of the change from former conditions caused by the reservoir work.

Attention is called to the fact that the slope of the excavation varies but slightly from that of the original surface, and that the maximum depth of cutting occurs on the west slope and not in the bed of the ravine. For the most part, the actual deepening of the original ravine, for the entire length of both reservoirs, ranged between 8 and 18 ft. It may also be noted that the line of the slip at Reservoir No. 4, when first discovered, was near the level of the original bed of the ravine, (see page 333 and Fig. 2, Plate XX), and not 8 or more feet below that point, at the bottom of the newly excavated basin.

Mr. Adams is undoubtedly correct in assuming that the rocky ridge to the east of Reservoir No. 4 is a barrier of unquestioned stability; but, in view of the fact that the maximum recorded movement of the slide has been observed at a point opposite this ridge, where the bed of the slide was found to be practically on a level with the original bed of the ravine, its effectiveness as a barrier may well be questioned. The footing which held the slide in place at this point, in accordance with the theory of those who contend that the excavation of the reservoirs caused a movement of the slide, or a resumption of such movement, could have been but a very few feet in depth, or, in fact, so slight a depth as to awaken a doubt as to its acting as an effective stop at any time.

Mr. Adams' second assumption, that the streams flowing in the ravines in which the reservoirs were built were not of an erosive character, may also be questioned. The writer has supposed that the existence of these ravines, having banks from 50 to 100 ft. high (see page 324), was due in part at least to the action of water at some period possibly remote. He has also a vivid recollection of the caution given

Mr. Clarke. him one day, soon after the reservoirs were built, by a citizen who had been familiar with the locality for a long term of years. This citizen's description of the torrent of water he had seen coming down the ravine, from melting snow on the hillside above, was not conducive to the writer's peace of mind, nor did he feel at ease until the artificial drainage system had been enlarged and improved.

It is also recalled that at one time during the winter of 1893-94, while the excavation for the reservoirs was in progress, an alarm was raised on behalf of the Sewer Department, and the Port of Portland Commission, who claimed that the grading contractor was sluicing a large amount of material into the Tanner Creek Sewer and thence to the river, where objectionable deposits were formed. These matters are mentioned merely as indicating that at certain seasons of the year the stream may have eroded its banks to some extent.

Mr. Adams is correct in his statement that the various depths of cutting at the reservoirs were less than 41 ft., but his assumption that the slopes of the excavation were increased in pitch beyond those of the original ravine is hardly supported by the profile, Plate XXVII. The 41-ft. cutting referred to by Mr. Adams was that for Shaft No. 21 (see page 366).

If the land surveys mentioned by Mr. Adams were made with care they should give reliable indications regarding the lack of movement during the period he names, but a slight movement might escape notice.

Regarding the railway track on Kingston Avenue, and the conditions existing there, which led Mr. Adams to conclude that the observed lateral movement was entirely of a local character, the writer calls attention to the profile of the cable railway, between Stations 7 and 15, covering the excavation and embankment where difficulty was experienced in keeping the track in proper alignment. It is not difficult to understand how the settlement of a 30-ft. embankment might throw the rails out of line, but why a shallow cutting should occasion a similar or even a greater distortion is not so evident. The earliest recorded surveys show the maximum movement of the track to have occurred in the shallow cutting near the 90° curve on Kingston Avenue (Fig. 1, Plate XIX), and the earliest and latest examinations have resulted in finding only signs of "buckling" of the track running east from that point, all rail joints being tightly closed and the rails lifted at vertical and horizontal bends (see Fig. 2, Plate XIX).

Mr. Adams expresses regret that the experts for the plaintiff were not allowed to review the testimony of the defendant and in consequence the essential point of difference in the expert opinion was obscured and lost on the jury. In his charge to the jury the trial judge said, in part:

"If this land was sliding in recent years before the excavation of the reservoirs, and would have been sliding since the excavation of

FIG. 1.—VIEW LOOKING EAST FROM RIDGE WEST OF SLIDE. THE WESTERN MARGIN OF SLIDE IS IN THE RAVINE IN THE FOREGROUND.

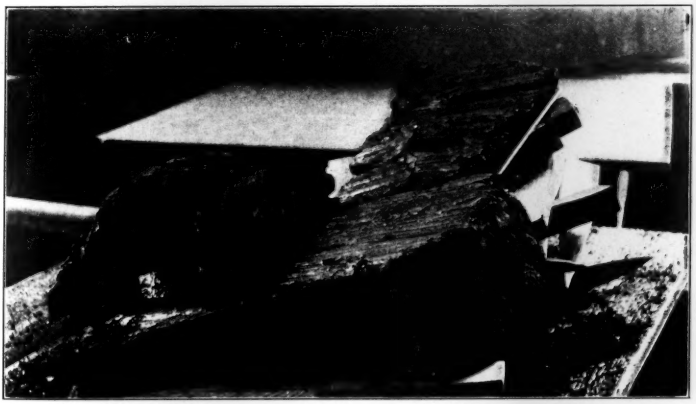


FIG. 2.—LUMP OF CLAY TAKEN FROM THE BOTTOM OF SHAFT NO. 5, 101 FEET BELOW THE SURFACE. THE LINE OF CLEAVAGE IS CLEARLY DEFINED.

the reservoirs, it would make no difference in plaintiff's claim or defendant's liability if it should be found that the slide was not continuous but intermittent. For instance, if it should be found that in the years immediately preceding the time of the excavation the land had slid in the winter and stood still in the summer, or had slid one year and had stood still the next, the city would not be liable though the land may have been at complete rest at the time of the excavation of the reservoirs and have started to move immediately thereafter; provided, that this movement was only a continuation of the intermittent movement which had existed before and which would have existed thereafter whether the reservoirs were constructed or not." Mr. Clarke.

The evidence submitted during the trial was sufficient to prove that the movement had at least been intermittent, if not continuous, and the members of the jury (all intelligent men of affairs, although not residing within the city limits) discerned the facts of the case, and, having done so, the only course open for them, under the instructions of the court, was to render a verdict for the City.

The writer is among those who believe that a close study of all the observed phenomena recorded in the paper will sustain the verdict as a most just and equitable one, and strictly in accordance with the facts of the case, notwithstanding the able argument in support of the opposite theory with which Mr. Adams closes his remarks. It is not believed that the verdict was due to any lack of skill on the part of the plaintiff's engineering experts, who made a brilliant effort to save the day for their clients, but they were doomed to defeat "to the satisfaction of the defendant and the grief of the plaintiff."

Mr. Schuyler has defined his official connection with the Portland Water-Works as being that of "Consulting Engineer," charged with the duty of designing and constructing the reservoirs. The writer, therefore, is very glad that attention has again been called to the careful manner in which the gate-houses and pipe connections were designed and constructed, so as to obviate any danger of interrupting the City's supply. The fact that for nearly ten years the City has had uninterrupted and satisfactory service, without the use of the reservoirs, is due to Mr. Schuyler's skillful design, and is a matter for sincere congratulation.

Regarding Mr. Schuyler's statement that several times during the season (1894) he had observed the distorted rails shown at the curve in the railway track (Fig. 2, Plate XIX), and had ascribed their condition to the local settlement and the slide below it, it may be stated that the slip first observed at Reservoir No. 4, at the point where the retaining wall and drainage tunnel were commenced, was between 500 and 600 ft. from the railway curve mentioned, a distance sufficient to make it more than a merely local settlement.

In reaching his conclusion that the slip was caused by the removal of the toe of the slide in excavating for the reservoirs, as indicated chiefly by the fact that the movement occurred in the dry season at

Mr. Clarke. the latter end of the summer, and that if it had "been moving all the year previous, the manifestation of it could not have been overlooked," Mr. Schuyler evidently forgets that the slip was observed within a very short time after the first section of the reservoir bank had been dressed down to a uniform slope, so as to make it possible to detect so slight a change. Prior to that time (see pages 332 and 377, the western slope of the reservoir had been in a rough and unfinished state, making it practically impossible to detect a movement of no greater magnitude than that which was then taking place.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 985.

GENERAL METHODS FOR THE CALCULATION OF STATICALLY INDETERMINATE BRIDGES, AS USED IN THE CHECK CALCULATIONS OF DESIGNS FOR THE MANHATTAN BRIDGE AND THE BLACKWELL'S ISLAND BRIDGE, NEW YORK.*

By Frank H. Cilley, S. B.

Two huge suspension bridges, the old "Brooklyn Bridge" and the "Williamsburg Bridge," now span the East River, uniting Manhattan with the adjacent boroughs of Greater New York on Long Island. Two other great bridges across the East River have been projected, the "Blackwell's Island Bridge" and the "Manhattan Bridge," for which designs were made by the City Department of Bridges, under the direction of Gustav Lindenthal, M. Am. Soc. C. E., former Commissioner of Bridges, assisted by a consulting architect, Mr. Henry F. Hornbostel.

The methods used in making check sets of stress calculations of these bridges, for which the writer was employed, is the subject of this paper. It is presented in the belief that the methods described are so exceptionally simple, so easy of application, so

* Presented at the meeting of September 21st, 1904.

general, as well as so accurate and reliable, that they are well worth the attention of American bridge engineers generally, and with the hope that the exceptional magnitude of the two great bridges to which these methods were applied with perfect success, will win for them attention and consideration which otherwise could hardly be secured. These methods include nothing new; that is. nothing not already well known and used in Europe; but, as far as the writer is aware, they are little known in America and have not previously been described in a complete, brief and clear manner. It is the purpose of the writer to attempt such a description, and he trusts that the reader will pardon any repetition of matter already well known, the inclusion of which is made necessary for the sake of completeness.

The two bridges in question are what are known as "statically-indeterminate structures," *viz.*, structures with more inner parts or outer restraints than are necessary for their definition (or to make them "just stiff"), and the calculation of the stresses, therefore, is not possible through the simple application of the equations of equilibrium. For this calculation there is required, in addition, the consideration of distortion under load and the consequent distribution of stress among the different parts of the structure. This involves, in turn, knowledge, not merely of the form of the structure, but of the dimensions, material and elastic qualities of its parts; and necessitates much longer and more complicated calculations, the results of which are much less reliable* than in the case of the simpler statically-determinate structures. However, these calculations are frequently necessary, as in this case, and we are solely concerned here with the exposition of a simple, clear and reliable general method for this purpose.

If the structure were determinate, *i. e.*, if it had no unnecessary outer restraints or inner parts, the calculation of the stresses by the ordinary laws of equilibrium would offer no difficulties. To make any actual indeterminate structure so calculable it would be necessary to remove the superfluous restraints and cut the superfluous parts. But in that case the structure would no longer act as it did previously.

*See "Some Fundamental Propositions Relating to the Design of Frameworks," *Technology Quarterly*, June, 1897; "Some Fundamental Propositions in the Theory of Elasticity,—a Study of Primary or Self-Balancing Stresses," *American Journal of Science*, April, 1901, and "The Exact Design of Statically Indeterminate Frameworks, An Exposition of its Possibility, but Futility," *Transactions*, Am. Soc. C. E., Vol. XLIII, p. 353.

Suppose, however, that the restraints thus removed and the stresses at the cut surfaces thus eliminated, be replaced by forces such as will make the distortion of the structure exactly what it would have been if the superfluous restraints had not been removed and the superfluous parts had not been cut. The stresses would then be maintained as though nothing had been changed, and they would readily be expressed in terms of the loads and the replacing forces. But these replacing forces are unknown. How shall they be determined? Simply by requiring them to be such that the distortions which they, together with the given loading, cause, shall be precisely the same throughout as though no restraints had been removed and no parts had been cut. That is, they must be such that the points free from restraint will, nevertheless, follow the path of restraint, and the cut surfaces of the parts cut shall, nevertheless, remain together as though they were not cut. These conditions we may express through equations of elastic distortion, and these equations will be found precisely equal in number to the unknown forces, and linear in form. Let us consider the particular cases of the Blackwell's Island and the Manhattan Bridges.

Both these bridges are frameworks, and are treated under the simplifying assumptions usual with frameworks, the calculation being limited, in both cases, to the main truss systems.

In the case of the Manhattan Suspension Bridge we have a structure of three spans to consider, *viz.*, one center span of 1 470 ft. and two side spans of 725 ft. each. The center span is an inverted arch supported on rocking towers. The side spans are inverted semi-arches, each supported by a tower at one end and an anchorage at the other. Horizontal motion of the anchor ends of the anchor spans is prevented by anchor chains, which are continuations of the top chords and which extend far down into the anchorage masonry, where they are made fast.*

Suppose one of the anchor chains cut; the structure would be reduced to three simple spans and become at once statically determinable. But it would not act as before. The cut anchor chain stress must be replaced by a pair of equivalent forces, applied to the

* The analysis of this structure as to the degree of indetermination of the outer forces may be made as follows: There are four points of support for the truss system; therefore, with two reaction elements at each support, there would be eight reaction elements to determine; but at the towers the reactions are vertical, thus leaving only six elements to be determined. For this, statics furnishes the three conditions of the equilibrium of forces in a plane and the two further conditions that the moment of all outer forces either side of a section at either tower, about the summit of that tower, is zero. One condition remains to be furnished through elastic relations.

cut ends. In order to determine this pair of forces, it is only necessary to express the condition that the cut ends of the chain shall remain just touching, as a result of the distortion of the structure from these forces and the load. Or, suppose that the middle lower chord bar of the center span be cut, that a hinge be introduced in the middle of the upper chord bar above it, and that suitable diagonals to this hinge be provided, in place of the normal pair of counters of the center panel. The structure is then reduced to an inverted three-hinged arch on two rocking piers braced by two anchored side spans, and, as before, becomes statically determined. Again, it would not act as before, and the stress in the cut bottom chord bar must be replaced by a pair of equal and opposite forces applied at the cut ends. In order to determine these forces it is only necessary to express the condition that the cut ends of the bar shall remain just touching, as a result of the distortion of the structure from these forces and the load.

In the case of the Blackwell's Island Bridge we have a cantilever structure of five clear spans to consider, *viz.:* (1) a Manhattan shore span of 469.5 ft.; (2) a cantilever arm of 591 ft. extending over the west channel of the East River, met by a similar cantilever arm of 591 ft. from the Blackwell's Island truss, making a river span of 1 182 ft.; (3) the main span of the Blackwell's Island truss over the island, 630 ft.; (4) a cantilever arm of 492 ft. over the east channel of the East River, met by a similar cantilever arm of 492 ft. from the Queens truss, making another river span of 984 ft.; and (5) the shore span of the Queens truss, of 459 ft. Unlike usual American cantilever bridges, which are of the statically determinate type, the ends of the cantilever arms are not here separated, horizontally, by a gap spanned by a connecting truss, but the end of the cantilever arm from one shore comes vertically over the end from the other shore, and the two are connected by a long, rigid, vertical member or "rocker arm." These two rocker arms, one in each river span, make the structure doubly statically indeterminate.*

* The analysis of this structure as to the degree of indetermination of the outer forces may be made as follows: There are six points of support of the truss system; therefore, with two reaction elements at each support, there would be twelve reaction elements to determine; but each shore span anchorage reaction is vertical and one of the Blackwell's Island truss reactions is also vertical, thus leaving only nine elements to be determined. For this, statics furnishes the three conditions of the equilibrium of forces in a plane, the two conditions that the moment of all outer forces on either side of a section through either rocker arm about any point of the arm is zero, and the two further conditions that the resultant horizontal component of all outer forces on either side of a section through either rocker arm is zero. Two conditions remain to be furnished through elastic relations.

Suppose both these rocker arms cut, the structure would be reduced to three simple spans with disconnected cantilever extensions and would become statically determinable. To make it act as before, the rocker arm stresses must be replaced by pairs of equal and opposite (unknown) forces applied to the cut ends. In order to determine these forces we must express the conditions that the cut ends of the bars shall remain just touching, as a result of the distortion of the structure from these forces and the load, two conditions (since there are two cut bars) corresponding to the two pairs of forces which are to be determined.

In all cases, then, the problem is reduced to that of expressing certain displacements of the structure in terms of a set of forces, some known and some unknown, acting upon it as a statically determinate structure. We may, therefore, by the usual methods, and simply through the laws of equilibrium, express the stresses in all parts of the structure in terms of these forces, known and unknown, and thence, through the stress-strain relations for the given materials and for the given sections, express the strains. It only remains to express the displacements sought in terms of these strains, equate them properly and obtain the required conditions. Thus far, probably nothing has been stated that is not familiar to all. But how shall we express the desired displacements in terms of the strains of the parts?

Various methods may be adopted to this end. Until quite recently, methods were applied which were either accurate, but very laborious, or, less laborious, but only approximate, often based on theorems of flexure. But, not much more than twenty or thirty years ago, it was noted that the "principle of virtual velocities" could be applied to this end. Briefly stated, this theorem simply asserts that the displacement of a point, A, of an elastic body, in a given direction, resulting from a given displacement or distortion of some other part of the body, is equal to the work done at that other part of the body by the stresses there due to unity force at A in the direction of its displacement, through their displacements. Specifically, if a bar, subject only to direct stress, be extended by Δ, and unity force, applied at A in the direction in which its (A's) displacement was desired, caused a stress, S', in the bar, then $S'\Delta$ is the displacement of A in the given direction caused by the extension, Δ. The basis of this statement is very simple. The work done by the force unity at

A, through its displacement, must equal the corresponding work done by the stress, S', through the stretch of the bar, since these are forces of a system in equilibrium, and the only ones whose relative displacement is involved. This means, of course, that the structure must be such that the independent extension of the bar is possible; in a word, it must be statically determined to that extent. Thus, to get the displacement of A in a given direction we have only to apply at A an imaginary force of unity in that direction, determine the corresponding stresses in the various parts of the (artificially) statically determined structure, find the work (positive or negative) done by these through the actual strains, find the algebraic sum of these work elements, and the result will be the actual displacement of the point, A, in the given direction, as a result of the given set of strains. Analytically, this is expressed for a framework in the well-known formula, $\delta = \Sigma\, S'\, \Delta$, in which S' expresses the stresses in the bars due to unity force at A in the direction of its required displacement, δ, and Δ expresses the actual elongations of the corresponding bars from whatever cause, as stress, temperature change, etc. The elongations, Δ, due to stress are, for prismatic bars of perfectly elastic material, of course,

$$\Delta = \frac{S\,l}{E\,a};$$

and the stresses, S, may be expressed as simple linear functions of the loads, L, and the unknown forces replacing the removed restraints and stresses. Ultimately, this evidently results in the expression of each δ (and therefore of each of the required conditions), in terms of a simple linear equation of the forces known and unknown. The solution of these linear equations, which are equal in number to the unknown forces, gives their values.*

Turn again to the particular examples to be considered. Suppose, in the Manhattan Bridge, that we cut an anchor chain and that we made the horizontal component of the stress in the anchor chain, H, the unknown force to be determined. Suppose we cut the Manhattan anchor chain at the shore end of the Manhattan shore span. Let us find the analytic condition that under any given load and the corresponding H, the separation of the cut ends of the anchor chain shall be zero. Or, still better expressed, in this case, let

* See George F. Swain—"On the Application of the Principle of Virtual Velocities to the Determination of the Deflections and Stresses of Frames." *Journal* of the Franklin Institute, Feb., 1883.

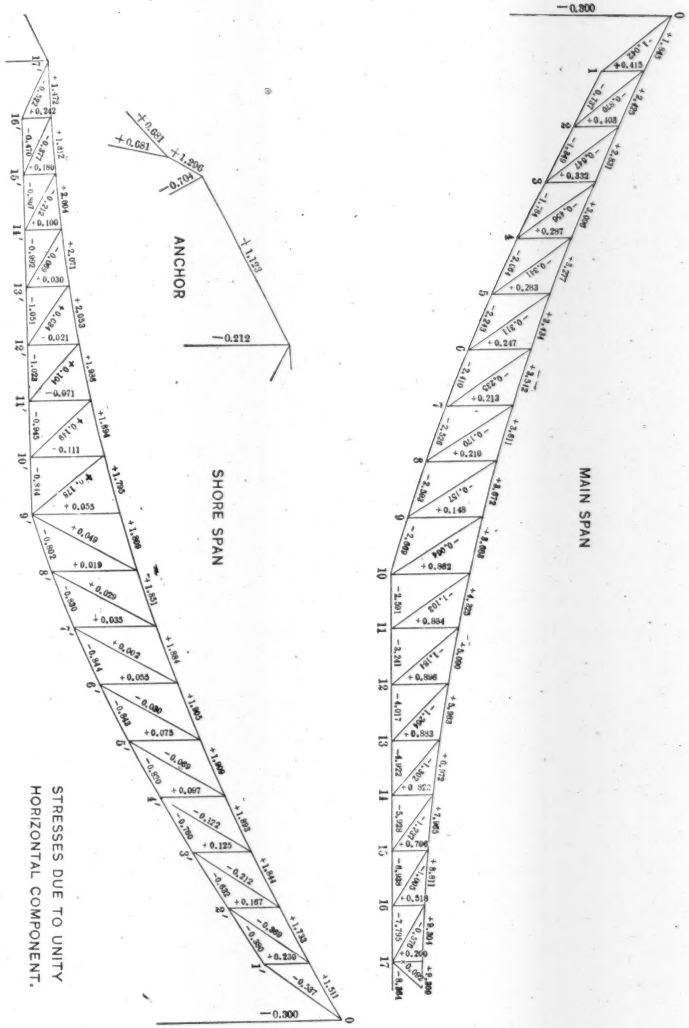

MAIN SPAN
SHORE SPAN
ANCHOR
STRESSES DUE TO UNITY
HORIZONTAL COMPONENT.

FIG. 1.

us find the separation of the cut ends of the anchor chain due to any given load, and the horizontal pull, H, necessary to reduce this to zero. This H will then be the required value.

First, let us find how much separation of the cut ends a horizontal pull of unity would overcome. Let S' be the stresses throughout the structure resulting from the application of a pair of equal and opposite pulls whose horizontal component is unity, on the cut ends of the cable—that is, the stresses in the structure due to unity "horizontal component" (see Fig. 1). The elongations of the bars under these stresses would be,

$$\Delta_{H=1} = \frac{S' l}{E a};$$

and the separation which would be overcome by $H = 1$, would be,

$$\delta_{H=1} = \Sigma S' \Delta_{H=1} = \Sigma \frac{(S')^2 l}{E a}.$$

What would be the separation of the cut ends of the anchor chain due to a given load acting alone? What, for example, would be the separation due to a load of unity applied at the center of the center span? Let S'' be the stresses throughout the structure due to unity load at that point. It must be remembered, here, that we are dealing with the structure with an anchor chain cut, and therefore acting as three simple spans. The S'' are simply the stresses in the middle span and the towers, due to load unity at the center of the middle span acting as a simple truss supported at its ends. The consequent elongations of the bars are,

$$\Delta'' = \frac{S'' l}{E a},$$

and the separation of the cut ends of the anchor chain resulting from these elongations is,

$$\delta'' = -\Sigma S' \Delta'' = -\Sigma S' \frac{S'' l}{E a},$$

the minus sign being used because the S' are due to unity pull, that is, they are forces acting contrary to separation. And since unity horizontal component will overcome $\delta_{H=1}$ separation, the actual horizontal component due to a load of unity at the center of the center span of the actual structure (with uncut anchor chains) must be,

$$H'' = \frac{\delta''}{\delta_{H=1}} = -\frac{\Sigma S' \frac{S'' l}{E a}}{\Sigma \frac{(S')^2 l}{E a}}.$$

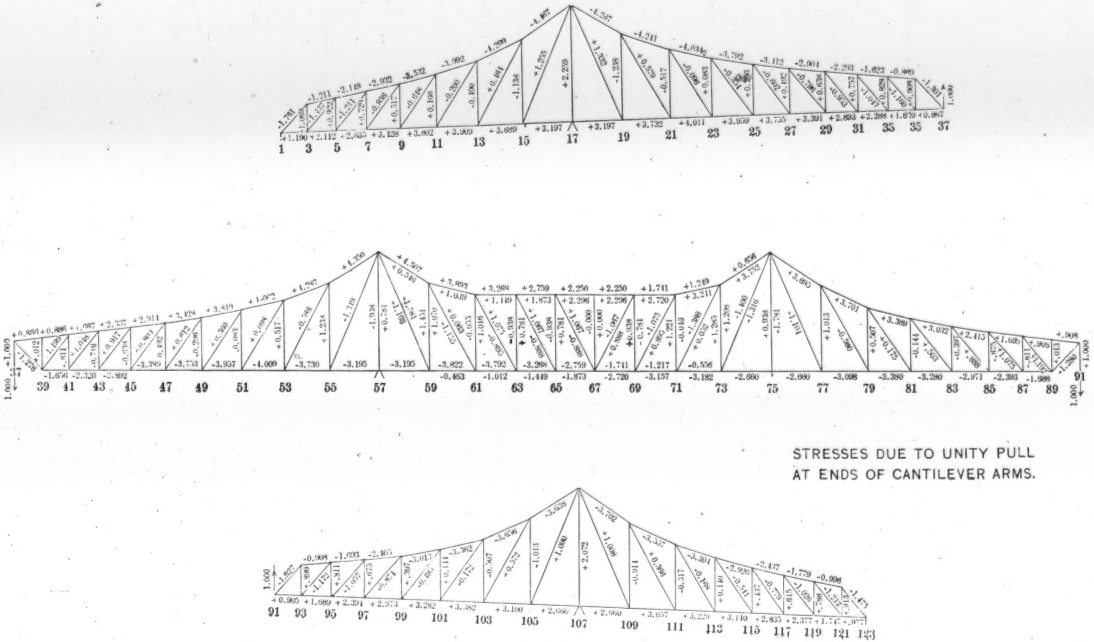


FIG. 2.

In the same way, by applying a load of unity at any other point, determining the consequent stresses and thence the strains of the members of the structure (with anchor chain cut), and so the consequent separation, δ, of the cut ends of the anchor chain, we could find the value of the horizontal component

$$H = \frac{\delta}{\delta_{H=1}},$$

necessary to reduce this separation to zero, that is, the real value of H due to this load on the actual structure. Thus we could find the value of H in the actual structure for load unity at each load point, whence, by multiplication by any loads, and summation, the value of H for those loads would follow. And since the actual stress due to any loading will be the stress for that loading, if there were no horizontal component (anchor chain supposed cut), plus the stress due to the actual horizontal component for that loading, or analytically,

$$S = S_o + H S';$$

the problem of the determination of the stresses in the actual structure for any actual loading would herewith be solved.

In the Blackwell's Island Bridge, suppose we had cut the two "rocker arms" and made their stresses, P_L, and, P_R, the unknown forces to be determined. The principles are the same as before, but the work is a trifle less simple, as we have two (unknown) variables instead of one. We have to find the analytic conditions that, under any given load and the corresponding rocker arm stresses, the separation of the cut ends of the rocker arms shall be zero.

First, let us find what displacements of the ends of the cantilever arms (the cut ends of the rocker arms) would result from unity pull at those ends, in the sense of tension in the rocker arms, but on the supposition that both rocker arms are cut. We have, in Fig. 2, the stresses due to such pulls of unity throughout the structure. Let S'_L denote these stresses in the left-hand or Manhattan truss, S'_{LC} those in the center or Blackwell's Island truss for pull of unity on the left (or Manhattan) cantilever end, S'_{RC} those in this truss for pull of unity on the right (or Queens) cantilever end, and S'_R those in the right-hand or Queens truss. The upward deflection of the end of the cantilever of the Manhattan truss, for any elongations, Δ_L, of its

bars would be, $\delta_L = \Sigma\, S'_L\, \Delta_L$, and, consequently, for the elongations

$$(\Delta_L)_{P_L=1} = \frac{S'_L\, l}{E\, a},$$

due to unity upward pull at the end of the cantilever arm, this deflection is,

$$(\delta_L)_{P_L=1} = \sum \frac{(S'_L)^2\, l}{E\, a};$$

similarly, the downward deflection of the end of the left cantilever arm of the center span, for any elongations, Δ_C, of the center span bars, is $\delta_{LC} = \Sigma\, S'_{LC}\, \Delta_C$; and so, for the elongations,

$$(\Delta_{LC})_{P_L=1} = \frac{S'_{LC}\, l}{E\, a},$$

due to unity downward pull at the end of the left cantilever arm, the downward deflection is,

$$(\delta_{LC})_{P_L=1} = \sum \frac{(S'_{LC})^2\, l}{E\, a}.$$

In the same way, for the downward deflection of the end of the right cantilever arm of the center span, we have, $\delta_{RC} = \Sigma\, S'_{RC}\, \Delta_C$, generally, and

$$(\delta_{RC})_{P_R=1} = \sum \frac{(S'_{RC})^2\, l}{E\, a},$$

for that due to unity downward pull at the end of the right cantilever arm, and for the upward deflection of the end of the cantilever of the Queens truss, we have, $\delta_R = \Sigma\, S'_R\, \Delta_R$, generally, and for unity upward pull at the end of the cantilever arm,

$$(\delta_R)_{P_R=1} = \sum \frac{(S'_R)^2\, l}{E\, a}.$$

Finally, the downward deflection of the end of the left cantilever arm of the center span, due to a downward pull of unity on the end of the right cantilever of the center span, is,

$$(\delta_{LC})_{P_R=1} = \sum \frac{S'_{LC} S'_{RC}\, l}{E\, a} = (\delta_{RC})_{P_L=1},$$

which is equal to the deflection of the end of the right cantilever arm of the center span, due to a pull of unity on the end of the left cantilever arm.

Now, let us raise the question, what would be the rocker arm

stresses due to a single load, say of unity, applied to any load point of the structure? Let these stresses be P_L and P_R, respectively. Then the actual stresses in the structure may evidently be built up of two parts, S_O, those due to the load alone, considering the rocker arms as cut, and $P_L\ S'_L$ in the left shore span, $P_L\ S'_{LC} + P_R\ S'_{RC}$ in the center span and $P_R\ S'_R$ in the right shore span. The corresponding elongations of the bars are

$$\frac{S_O\, l}{E\, a},\ P_L \frac{S'_L\, l}{E\, a},\ P_L \frac{S'_{LC}\, l}{E\, a} + P_R \frac{S'_{RC}\, l}{E\, a},\ \text{and}\ P_R \frac{S'_R\, l}{E\, a}.$$

The consequent separation of the cut ends of the left rocker arm will be

$$O = \delta_L + \delta_{LC} = P_L \left[\sum \frac{(S'_L)^2\, l}{E\, a} + \sum \frac{(S'_{LC})^2\, l}{E\, a} \right]$$

$$+ P_R \sum \frac{S'_{LC}\, S'_{RC}\, l}{E\, a} + \begin{cases} \sum \frac{S'_L\, S_O\, l}{E\, a} \\ \text{or} \\ \sum \frac{S'_{LC}\, S_O\, l}{E\, a}, \\ \text{or} \\ O \end{cases}$$

according as the load is on the left, the center or the right truss

Similarly, the separation of the cut ends of the right rocker arm will be:

$$O = \delta_{RC} + \delta_R = P_L \sum \frac{S'_{LC}\, S'_{RC}\, l}{E\, a}$$

$$+ P_R \left[\sum \frac{(S'_{RC})^2\, l}{E\, a} + \sum \frac{(S'_R)^2\, l}{E\, a} \right] + \begin{cases} O \\ \text{or} \\ \sum \frac{S'_{RC}\, S_O\, l}{E\, a}, \\ \text{or} \\ \sum \frac{S'_R\, S_O\, l}{E\, a} \end{cases}$$

according as the load is on the left, the center or the right truss. In these expressions, the terms for the stretch of the rocker arms must not be forgotten. The solution of these two simultaneous equations which are linear in our only two unknown quantities, P_L, and, P_R, will give their values. By solving these equations, successively, for a load unity at each load point, we could find the values of the rocker

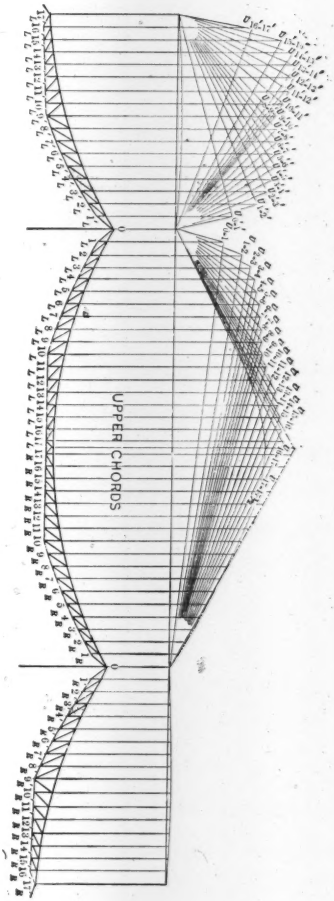


Fig. 3.

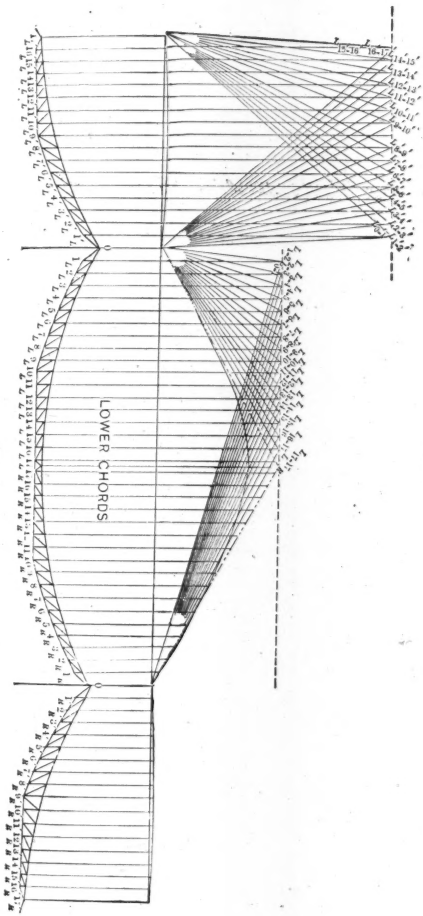


Fig. 4.

arm stresses for each such load, and thence, by summation, for any loads. The actual stresses due to any loading are those which would exist if there were no rocker arms plus those due to the actual rocker arm stresses, or, analytically,

$$S = S_0 + P_L S'_L, \quad S = S_0 + P_L S'_{LC} + P_R S'_{RC}, \quad S = S_0 + P_R S'_R,$$

left truss center truss right truss

so that the problem for the actual structure and for any loading is thus solved.

We may note that, for any loading, the equations

$$0 = \delta_L + \delta_{LC} = P_L \left[\sum \frac{(S'_L)^2 l}{E a} + \sum \frac{(S'_{LC})^2 l}{E a} \right]$$

$$+ P_R \sum \frac{S'_{LC} S'_{RC} l}{E a} + \sum \frac{S'_L S_0 l}{E a} + \sum \frac{S'_{LC} S_0 l}{E a},$$

and

$$0 = \delta_{RC} + \delta_R = P_L \sum \frac{S'_{LC} S'_{RC} l}{E a}$$

$$+ P_R \left[\sum \frac{(S'_{RC})^2 l}{E a} + \sum \frac{(S'_R)^2 l}{E a} \right] + \sum \frac{S'_{RC} S_0 l}{E a} + \sum \frac{S'_R S_0 l}{E a},$$

furnish directly, by their solution, the values of P_L and P_R and thence the actual stresses. We may further note the special cases, first, of a load of unity suspended on the end of the left (or Manhattan) truss cantilever arm, for which $S_0 = -S'_L$, simply, where we find by solution the rocker arm stresses,

$$(P_L)_L = \frac{\sum \frac{(S'_L)^2 l}{Ea}\left[\sum \frac{(S'_R)^2 l}{Ea} + \sum \frac{(S'_{RC})^2 l}{Ea}\right]}{\left[\sum \frac{(S'_L)^2 l}{Ea} + \sum \frac{(S'_{LC})^2 l}{Ea}\right]\left[\sum \frac{(S'_R)^2 l}{Ea} + \sum \frac{(S'_{RC})^2 l}{Ea}\right] - \left[\sum \frac{S'_{LC} S'_{RC} l}{Ea}\right]^2},$$

and

$$(P_R)_L = \frac{-\sum \frac{(S'_L)^2 l}{Ea} \sum \frac{S'_{LC} S'_{RC} l}{Ea}}{\left[\sum \frac{(S'_L)^2 l}{Ea} + \sum \frac{(S'_{LC})^2 l}{Ea}\right]\left[\sum \frac{(S'_R)^2 l}{Ea} + \sum \frac{(S'_{RC})^2 l}{Ea}\right] - \left[\sum \frac{S'_{LC} S'_{RC} l}{Ea}\right]^2};$$

and, second, the case of a load of unity suspended on the end of the right (or Queens) truss cantilever arm for which $S_O = -S'_R$, simply, and where,

$$(P_L)_R = \frac{-\sum \frac{(S'_R)^2 l}{Ea} \sum \frac{S'_{LC} S'_{RC} l}{Ea}}{\left[\sum \frac{(S'_L)^2 l}{Ea} + \sum \frac{(S'_{LC})^2 l}{Ea}\right]\left[\sum \frac{(S'_R)^2 l}{Ea} + \sum \frac{(S'_{RC})^2 l}{Ea}\right] - \left[\sum \frac{S'_{LC} S'_{RC} l}{Ea}\right]^2},$$

and

$$(P_R)_R = \frac{\sum \frac{(S'_R)^2 l}{Ea}\left[\sum \frac{(S'_L)^2 l}{Ea} + \sum \frac{(S'_{LC})^2 l}{Ea}\right]}{\left[\sum \frac{(S'_L)^2 l}{Ea} + \sum \frac{(S'_{LC})^2 l}{Ea}\right]\left[\sum \frac{(S'_R)^2 l}{Ea} + \sum \frac{(S'_{RC})^2 l}{Ea}\right] - \left[\sum \frac{S'_{LC} S'_{RC} l}{Ea}\right]^2};$$

We have now seen how the calculation of an indeterminate structure may be reduced to that of a determinate structure, by doing away with superfluous restraints and cutting superfluous parts, substituting for the influences thus destroyed corresponding replacing forces; how these replacing forces are determined by expressing the condition that the distortions shall remain as before; and how we may most easily express these latter conditions, analytically, by the aid of the principle of virtual velocities, thus obtaining simultaneous linear equations in the unknown replacing forces (equal in number to those forces), whose solution gives the values of these forces and so reduces the calculations to those of determinate structures. And we have followed this process in the case of two particular structures. But much yet remains to be considered in the way of perfecting and completing our procedure.

We have, it is true, already shown how we may obtain an analytic expression for the value of each of the replacing forces for any given loading, and we have given such expressions, or their elements, for the two bridges whose calculation is here used for illustration. But, for such calculation, we wish to be able to determine readily these replacing forces for any loading, and the application of these formulas separately to each loading would be exceedingly laborious, for even a single application involves much labor. Evidently, anything that diminishes this labor will be highly desirable, and, as a first step, we must note the determination of the values of the replacing forces for a load of unity at each of the load points. Then the values for all other loadings follow at once from these by simple multiplication and summation. This procedure would reduce the applications of our formulas to as many loadings as there are load points only, a great saving, but still leaving far too much work to be done. Here, again, another theorem comes to our rescue—that of the reciprocal nature of displacements, often called, in the theory of structures, "Maxwell's reciprocal theorem," and known, in the general theory of the elasticity of solid bodies, as "Betti's reciprocal theorem." Briefly, Maxwell's theorem asserts that the displacement of any point, A, in any given direction, $A\ C$, due to unity force at B in any given direction, $B\ D$, is equal to the displacement of B in the direction, $B\ D$, due to unity force applied at A in the first direction, $A\ C$. The reason for this is exceedingly simple. Let the displacement of A in

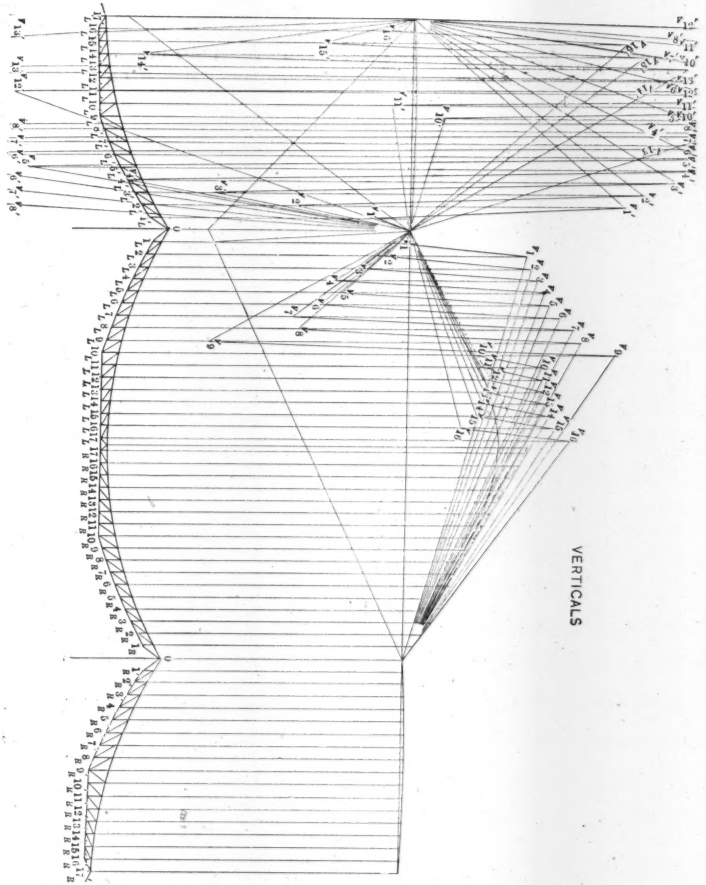


FIG. 5.

the direction, $A\ C$, due to unity force at B be a, and the simultaneous displacement of B in the direction, $B\ D$, of the force be b'. The potential energy of the body will be increased through the application of the force of unity at B by $\frac{1}{2}\ b'$. Now let unity force be applied at A in the direction, $A\ C$, of the displacement, a, and let A suffer the consequent further displacement, a', in the same direction, and B the further displacement, b, in the same direction, $B\ D$, as b'. The potential energy will be further increased by $\frac{1}{2}\ a' + b$, or the total increase from applying both forces of unity will be,

$$\Delta\ E_1 = \frac{1}{2}\ a' + \frac{1}{2}\ b' + b.$$

But suppose the forces of unity had each been applied, first at A and then at B. A, under the force there applied, would have been displaced by a', while B was simultaneously displaced by b, the potential energy now increasing $\frac{1}{2}\ a'$. Then, applying the second force of unity at B, it would be displaced by b', while A would simultaneously be displaced by a, resulting in a further increase of the potential energy of $\frac{1}{2}\ b' + a$, and the total increase, from applying both forces in this order, would be $\Delta\ E_2 = \frac{1}{2}\ a' + \frac{1}{2}\ b' + a$. But since the order of application of the loads cannot affect the ultimate result, $\Delta\ E_1 = \Delta\ E_2$, and therefore $a = b$.*

Consider the practical bearing of this theorem on our problem. We wish to know the stress in a certain superfluous member or the restraint acting on a certain part of the structure due to forces applied at any of certain other points which we may term load points. Suppose the member, whose stress is sought, to be cut, or the restraint, whose amount is desired, to be removed (all other superfluous members or restraints remaining unchanged), and a pair of equal and

* The following brief proof based on the principle of virtual velocities is recommended by Prof. George F. Swain, M. Am. Soc. C. E. Let S'_A be the stresses in the bars due to unity force at A in the direction, $A\ C$, and S'_B the stresses in the bars due to unity force at B in the direction, $B\ D$. The displacement of B in the direction, $B\ D$, due to unity force at A in the direction, $A\ C$, will then be, $b = \sum \frac{S'_B\ S'_A\ l}{E\ a}$. But the displacement of A in the direction, $A\ C$, due to unity force at B in the direction, $B\ D$, will be, $a = \sum \frac{S'_A\ S'_B\ l}{E\ a}$, whence $a = b$.

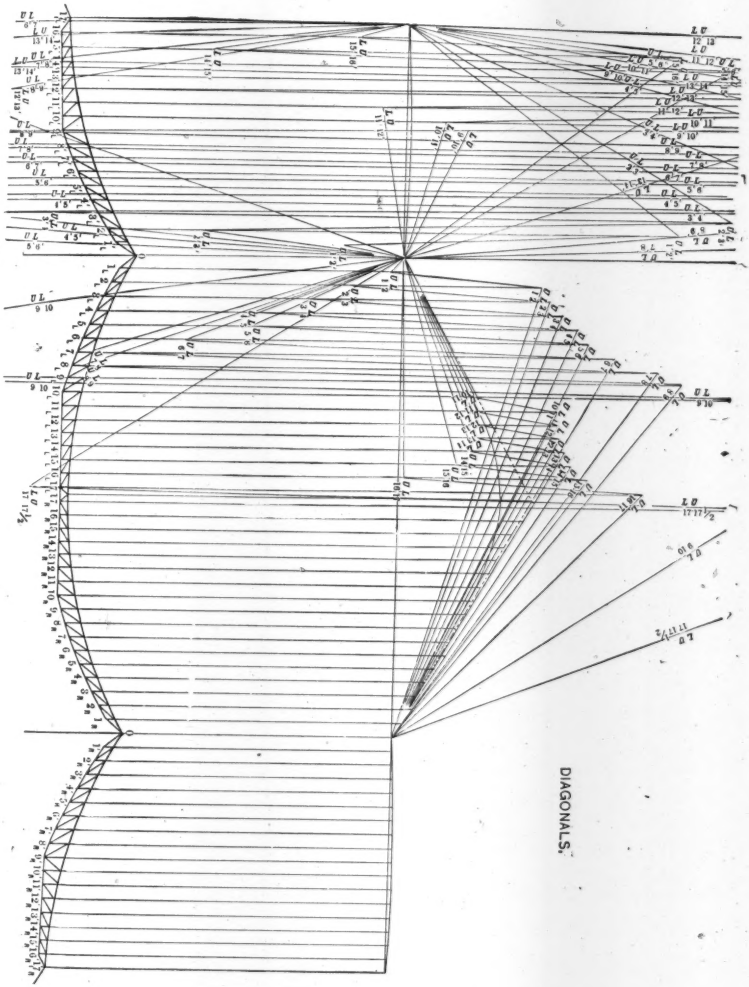


FIG. 6.

opposite forces of unity to be applied at the cut ends or in place of the removed restraint. Through the methods already described, we can determine the consequent stresses, and therefore strains, of all members due to the pair of forces of unity. Suppose we now determine the consequent displacements of all the load points and also the separation of the ends of the cut member, or of the points between which the removed restraint acted. Evidently, by Maxwell's theorem, the separation of the ends of the cut member (or of the points between which the removed restraint acted) due to unity force applied at any load point, would be equal to the displacement of that load point in the direction of this applied force, due to the pair of forces of unity. And this quantity, divided by the separation of the cut ends (or of the points between which the removed restraint acted), due to the pair of forces of unity, would give the value of the stress or restraining force sought.

The problem is now reduced to that of obtaining a displacement diagram of the load points for a known set of strains. There are many methods by which this may be done, among others that of the successive application of the principle of virtual velocities to each load point; but even this is unnecessarily long and laborious, unless exceptional accuracy is desired. Far simpler and briefer, in the case of frameworks, are certain graphical methods which have now to be described.

The first of these is the Williot displacement diagram. Suppose that from any two points, A and B (Fig. 7), two bars, $A\ C$ and $B\ C$, extend, meeting in C. Suppose that A and B each suffer certain displacements, and, further, that $A\ C$ and $B\ C$ suffer certain changes in length, and that we have to find the resulting displacement of the joint, C.

Imagine $A\ C$ and $B\ C$ freed at C. Let $A\ A'$ be the given displacement of A, and $B\ B'$ that of B. Conceive $A\ C$ moved parallel to itself to $A'\ C'_A$, and, similarly, $B\ C$ moved to $B'\ C'_B$. Let $C'_A\ C''_A$ be the change in length of $A\ C$, and $C'_B\ C''_B$ that of $B\ C$. Evidently, if we now swing $A'\ C''_A$ about A', and $B'\ C''_B$ about B', until the free ends meet at C''', this will be the new position of the joint, and $C\ C'''$ will be the displacement sought. Practically the displacements, $A\ A'$ and $B\ B'$, and the changes in length, $C'_A\ C''_A$ and $C'_B\ C''_B$, will be extremely small compared with the lengths, $A\ C$ and $B\ C$, and the arcs

of circles, $C''_A\ C'''$ and $C''_B\ C'''$, will practically coincide with the perpendiculars from C''_A and C''_B to the directions of $A\ C$ and $B\ C$, respectively. Moreover, we note that the only essential part of the construction is the figure, $C\ C'_A\ C''_A\ C'''\ C''_B\ C'_B$, which is shown here alone in Fig. 8. This, then, is all we need draw. And, since exaggeration does not affect its proportions, we may make it of any convenient size. Thus, from a point, C, we lay out the displacements of A and B at a thousand times their real amount, or $C\ C'_A$ and $C\ C'_B$. From C'_A and C'_B we lay off, in the directions of the bars (which remain practically unchanged) their changes in length, $C'_A\ C''_A$ and $C'_B\ C''_B$, respectively, also exaggerated a thousand times. From C''_A and C''_B we draw lines, $C''_A\ C'''$ and $C''_B\ C'''$, perpendicular in direction to the directions of the bars, and their intersection, C''', furnishes us the displacement, $C\ C'''$, of C in amount (exaggerated a thousand times) and in direction. The method is exceedingly simple and, when the intersections are good, is exact. When the intersections are not good, the construction itself is likely to be bad.

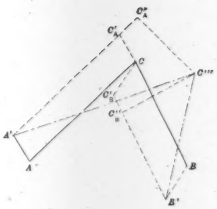


FIG. 7.

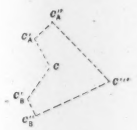


FIG. 8.

This method, applied repeatedly, enables us to find the displacement of all joints of any framework which can be built up by adding successive pairs of bars, for any given set of strains. We must start by assuming a fixed point and a fixed direction and build upon these. The various displacements, thus found, will then have to be corrected, generally, for the error in the assumption that a certain direction was fixed. This is most readily done by noting the true displacement of some point, which is usually obvious, and proportioning the corrections of all other points to its correction, a simple matter, since it involves simply the corrections due to a small rotation of the framework. The displacement of each joint, due to the rotation of the framework about any point, is, in direction, perpendicular to the line connecting the point with that joint and, in amount, proportional to the length of this connecting line. If, therefore, we draw a figure similar to our framework, but rotated 90°, in the sense of the correct-

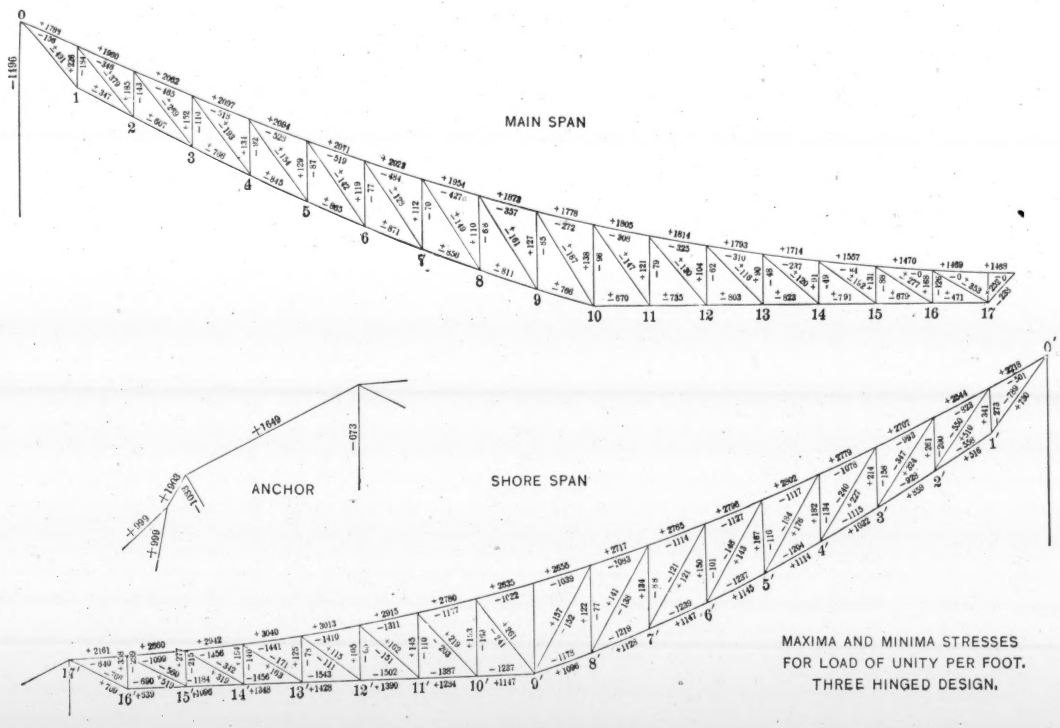
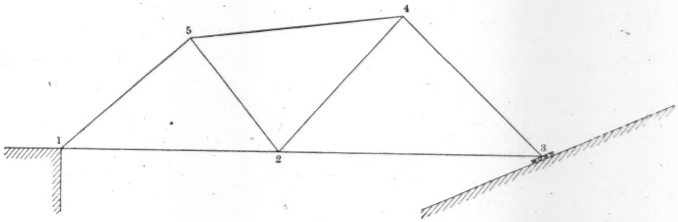

MAIN SPAN
ANCHOR
SHORE SPAN
MAXIMA AND MINIMA STRESSES
FOR LOAD OF UNITY PER FOOT.
THREE HINGED DESIGN.

FIG. 9.

ng rotation, with one of its points, which is actually fixed, coinciding with the displacement position of that point in the Williot diagram, and to such scale as shall make the line from any other of its points to the corresponding displacement position of that point in the Williot diagram correctly represent the actual direction of the displacement of that point, then the points of this figure, thus drawn, will give us, by their distances and directions from the actually fixed point, the



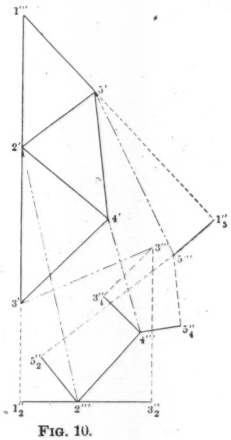


FIG. 10.

rotation displacements of all these points. The distance and direction from any one of the joints of this figure to the corresponding displacement position of the point in the Williot diagram will give the actual displacement in amount and direction (all, of course, to the exaggerated scale).

For example, consider the simple truss in Fig. 10 supported at 1 and on rollers at 3. Suppose we regard 2 as a fixed point and 2–4 as a fixed direction, and, starting from 2‴, proceed to the corresponding Williot displacement diagram. Suppose 2–4 to have the extension 2‴–4‴. The extension, 2‴–5″$_2$, of 2–5 and compression, 4‴–5″$_4$, of 4–5 enable us to find the corresponding displacement position, 5‴, of 5, whence the extension, 2‴–1″$_2$, of 2–1 and the compression, 5‴–1″$_5$, of 5–1 enable us to find the displacement position, 1‴, of 1. Lastly, the extension, 2‴–3″$_2$, of 2–3 and the compression, 4‴–3″$_4$, of 4–3 give us the displacement position, 3‴, of 3. That is to say, 2‴–1‴ represents the magnitude and direction of the motion of 1, 2‴–5‴ that of 5, 2‴–4‴ that of 4, and 2‴–3‴ that of 3, all with

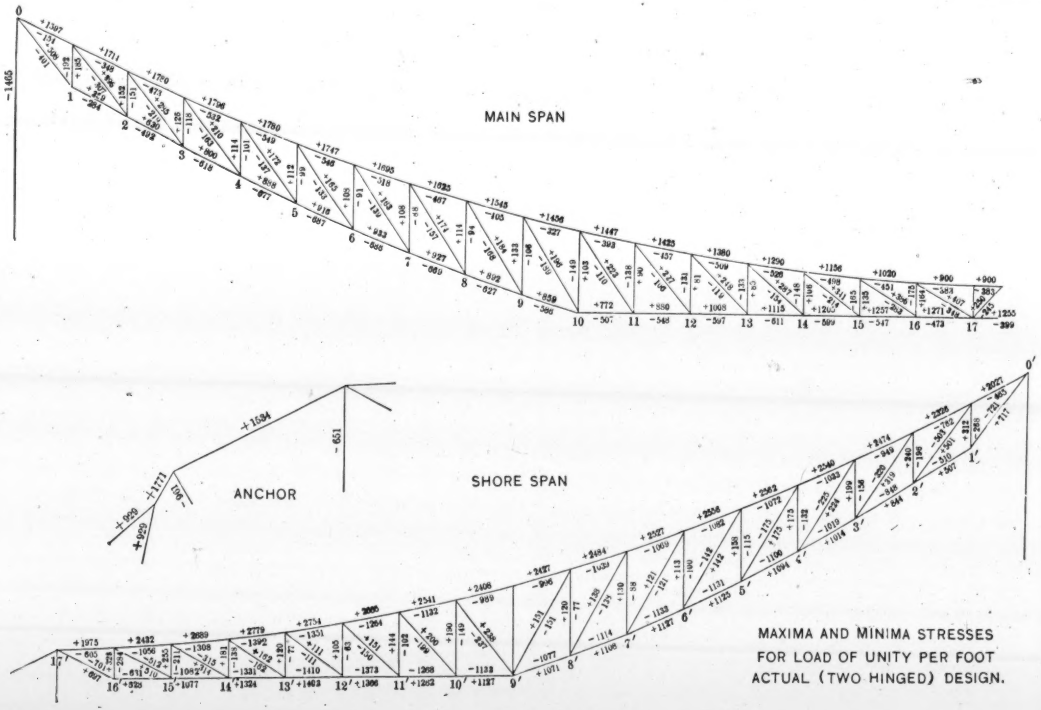

MAIN SPAN
ANCHOR
SHORE SPAN
MAXIMA AND MINIMA STRESSES
FOR LOAD OF UNITY PER FOOT
ACTUAL (TWO HINGED) DESIGN.

FIG. 11.

reference to 2 as a fixed point and 2–4 as fixed in direction. Now the actual displacement of 1 is zero, and the actual displacement of 3 is in the direction of the plane of its roller bed, instead of the direction, 2‴–3‴, just found. Thus, the actual displacement of 3 must be found by combining its displacement, 1‴–3‴, with respect to 1 on the supposition that 2–4 is a fixed direction, with such a rotational displacement about 1 as shall give the actual displacement in the roller bed direction. This rotational displacement of 3 occurs perpendicular to the line 1–3, therefore we find the intersection, 3′, of the line, 1‴–3′, perpendicular to 1–3 with the line, 3‴–3′, in the roller bed direction, and 3′–3‴ is the actual displacement of 3 in amount and direction. Now we construct the figure 1‴ 2′ 3′ 4′ 5′, similar to the original truss figure, 1 2 3 4 5. Then, 2′–2‴, 3′–3‴, 4′–4‴ and 5′–5‴, will be the actual displacements of the points, 2, 3, 4, and 5, in amount and direction (to the exaggerated scale).

The Williot displacement diagram thus furnishes an exceedingly simple and beautiful method of attaining the desired end. But, practically, this method has defects. It tends to sum up and multiply errors, even more than does the use of the ordinary Cremona stress diagram. And if the elongations of the members are laid out on a reasonable scale the displacement diagram as a whole becomes excessively large. By making several independent diagrams, especially with different points and lines assumed as fixed, and averaging the results, the first of these objections can be overcome in part, but for greater accuracy another procedure is often preferable.

In bridges, the load points are, as a rule, either the top or bottom chord joints or are connected, simply, with them. All that is needed in such a case, then, is the "deflection curve" of the "load line" for unity value of the replacing force under consideration. This curve permits of comparatively accurate and ready determination in a variety of ways, but, in the case of frameworks, probably the best way is, first, to determine the deflection angle at each "panel point" of the load line, from which the deflection curve readily follows. To determine these angles (say it is the angle between two adjacent chord members, in each case), we have to determine the distortion angles of the triangles

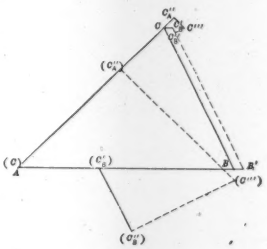


FIG. 12.

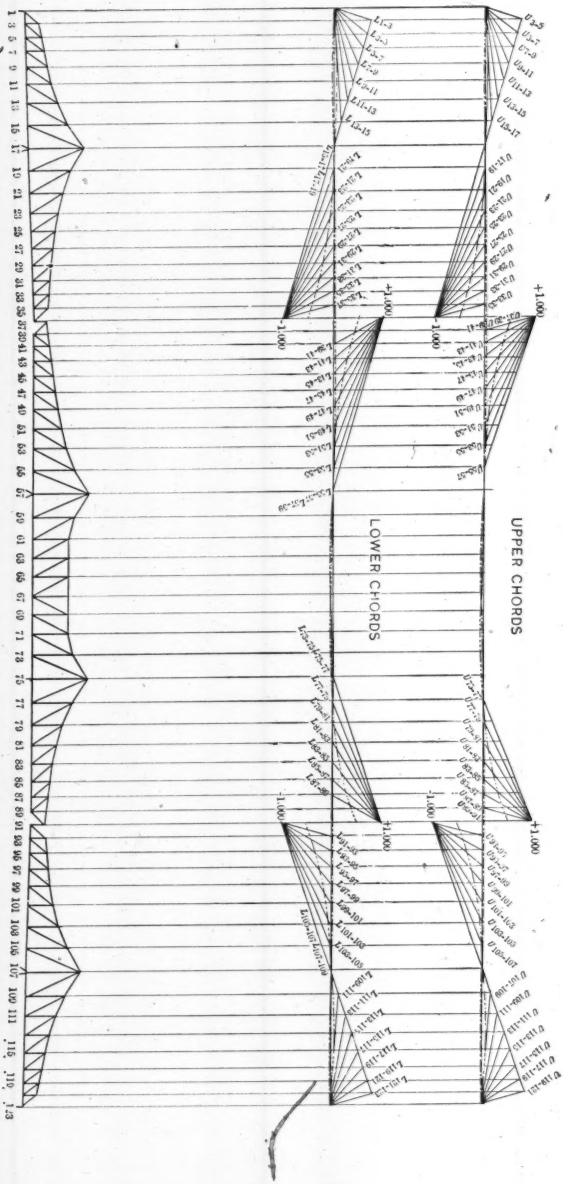


FIG. 18.

whose sides are in part the members meeting at the joint. And these distortion angles, we may very readily determine by the aid of Williot polygons, as follows: Let $A\ B\ C$ (Fig. 12) be such a triangle and the distortion angle at B for given elongations of $A\ B$, $B\ C$ and $A\ C$ be required. Let $A\ B$ increase by $B\ B'$; let $A\ C$ increase by $C\ C''_A$, and let $B\ C$ shorten by $C'_B\ C''_B$. Then C''' will be the resulting position of C. But this means that $B\ C$ will have swung through the small angle, $\frac{C''_B\ C'''}{B\ C}$. We have then only to determine, accurately, $C''_B\ C'''$, in order to solve the problem. Practically, we lay out $C\ C''_A$ and $B\ B' = C\ C'_B$ from A on the triangle, $A\ B\ C$, to an exaggerated scale, as $(C)\ (C''_A)$ and $(C)\ (C'_B)$. Then we draw $(C'_B)\ (C''_B)$ parallel to $B\ C$ and equal to $C'_B C''_B$ on the same scale. The perpen-

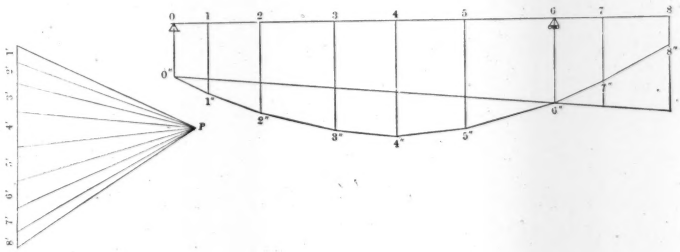


FIG. 14.

dicular to $A\ C$ through (C''_A) and that to $B\ C$ through (C'_B) give (C''') and thus $(C''_B)\ (C''')$ which, measured on the exaggerated scale, is the quantity sought. The triangle, $A\ B\ C$, on which this work is done, would preferably be a part of the figure of the truss, carefully and accurately drawn. Proceeding thus, from triangle to triangle, we readily obtain these distortion angles, and thus, by summation at each joint, carefully observing signs, the deflection angles at each joint. It now only remains to find the corresponding deflection curve.

The deflection angles thus being known, the deflection curve may be easily found, either graphically or analytically. The former method is a trifle simpler and, generally, is amply accurate. We proceed as follows: Suppose we have a straight horizontal load line, as is practically the case in most bridges. On a vertical we carefully lay

off successive distances proportional to the successive deflection angles, preferably by adding the figure values of the angles to one side and plotting the sums all from one point, to avoid the accumulation of errors. Now, if lines were drawn from these points to a point at a suitable (great) distance, the angles between them would be the deflection angles. Instead, we take a comparatively near point or "pole," by which the tangents of the angles are multiplied in a corresponding degree, and draw radiating lines to the set of points. Now, parallel to these lines from the pole, we draw, from panel vertical to panel vertical of the load line, lines of the deflection curve, which is thus at once obtained on the exaggerated scale. We have only to note on this curve the points of no deflection, draw straight lines through them, and from these measure the deflections. They will be the desired deflections to the exaggerated scale.

Thus, let 1, 2, 3, 4, 5, 6, 7, 8 (Fig. 14) be the panel points of the load line, and let 1′, 2′, 3′, 4′, 5′, 6′, 7′, 8′ be proportional to the deflection angles at these points. Let P, at a convenient distance, be taken as a pole and by its aid the curve (polygon), 1″ 2″ 3″ 4″ 5″ 6″ 7″ 8″, be drawn with its sides ("strings") parallel to the corresponding "rays" from P. This curve will be, to an exaggerated scale, the deflection curve sought. Let the points, 0 and 6, of the load line be points of no deflection; then the actual deflections must be measured from the line, 0″ 6″, to the points of the deflection curve. Questions of scale must be carefully looked into, of course, but this is a matter of detail for which we are not warranted in taking space here.

This completes the description of the general method to be pursued in the complete determination of the replacing forces for all load points. Let us briefly note what this work was in the cases of the Manhattan and the Blackwell's Island Bridges.

In the case of the Manhattan Bridge we had the value of but one replacing force to determine for each load point, namely, the value of the "horizontal component." Cutting the anchor chain and applying a pair of tensions, of unity horizontal component, to the cut ends, a set of stresses (and therefore strains) resulted, which we have already noted, and, consequently, a certain deflection curve of the load line. This was found by determining, first, the distortion angles of each of the triangles which formed the trusses, resulting from the strains due to unity horizontal component, a Williot polygon being drawn for

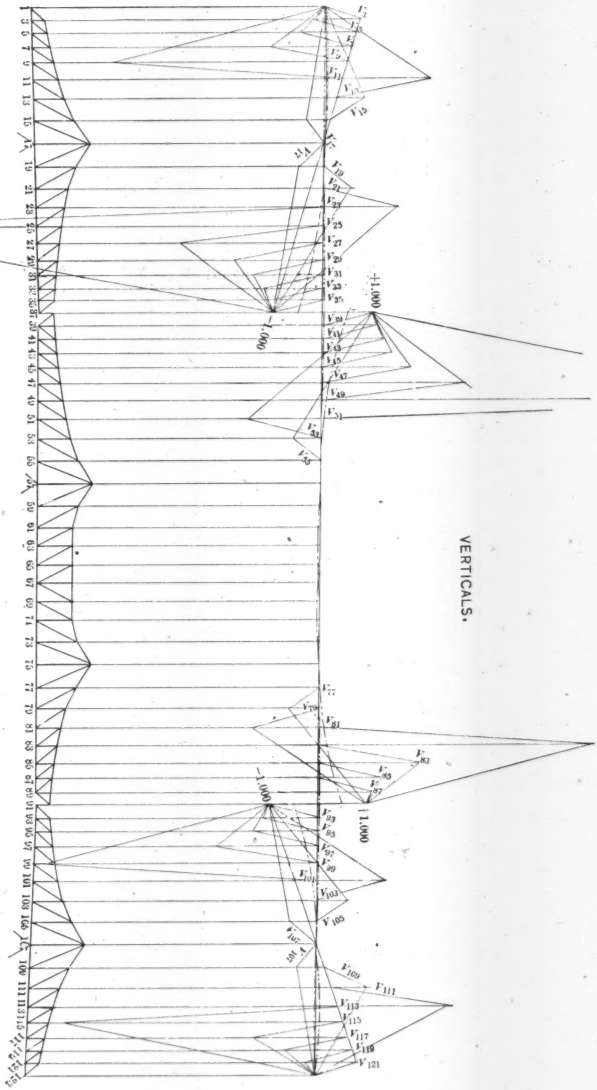


FIG. 15.

each triangle. Thence the deflection angles of the bottom chords were found by simple summation of the distortion angles at each joint. Thence the deflection curve of the bottom chord was found, which would be the same, in this case, as that of the load line. But, as the bottom chord was, in some parts, neither practically straight, nor horizontal, it was necessary that certain small corrections (equal in each panel to the vertical component of the shortening of the chord bar in that panel) be added to the deflections which were found solely from the deflection angles as hitherto described. The final result was a load-line deflection curve for each of the side spans and the center span, whose ordinates were strictly proportional to the value of the horizontal component in the bridge due to load of unity at those points. Dividing these ordinates by the separation of the cut ends of the anchor chain, which unity horizontal component would overcome, previously analytically determined, the actual values of H for load unity at the various panel points were obtained. These values, plotted, gave what is known as the "H curve" or "influence line" for H, which is shown in the illustrations for the influence lines.

In the case of the Blackwell's Island Bridge the procedure was quite similar, but a trifle less simple, for there were two indeterminate forces to consider. The deflection angles and curves were determined, as before, for each main panel point, and for unity pull at the end of each cantilever arm. Now, the left rocker arm alone being supposed cut, and a pair of equal and opposite pulls of unity applied to the cut ends, a certain consequent stress, $\frac{(P_R)_L}{(P_L)_L}$, resulted in the right rocker arm, and the actual deflections were as follows: For the Manhattan truss, those due to unity upward pull at the left rocker arm; for the Blackwell's Island truss those due to unity downward pull on the left rocker arm plus those due to $\frac{(P_L)_R}{(P_L)_L}$ on the right rocker arm; and, finally, for the Queens truss, those due to $\frac{(P_R)_L}{(P_L)_L}$ upward pull on the right rocker arm. The influence line ordinates of the left rocker arm stress were proportional to these ordinates, and since the actual values for load unity at the left and the right rocker arm points were $(P_L)_L$ and $(P_R)_L$, all other values thence followed at once. The procedure for the influence line for the right

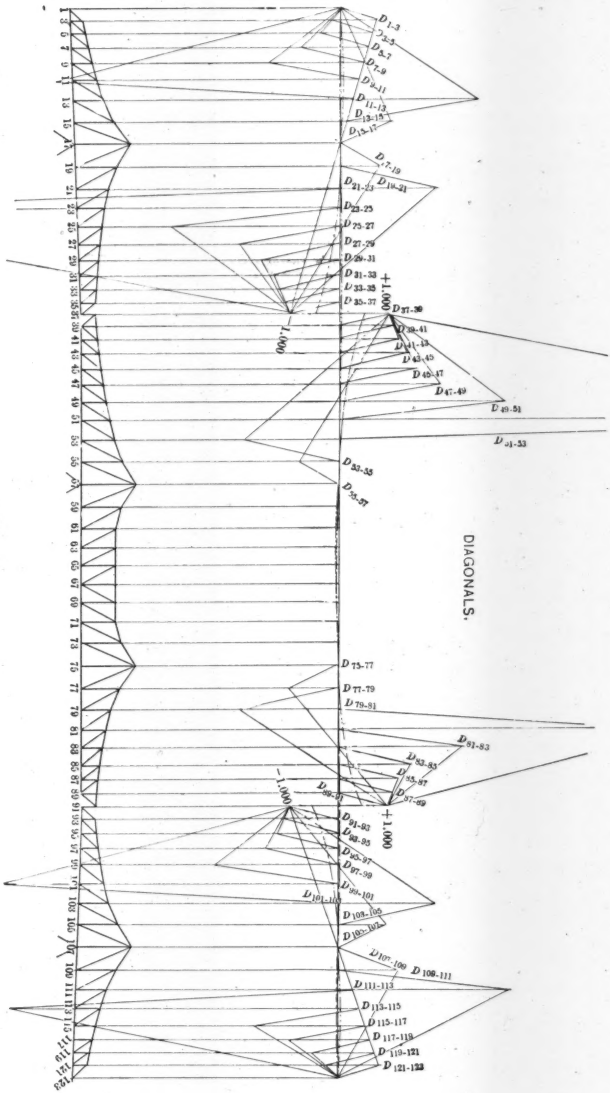

D 1-3
D 3-5
D 5-7
D 7-9
D 9-11
D 11-13
D 13-15
D 15-17
D 7-19
D 19-21
D 21-23
D 23-25
D 25-27
D 27-29
D 29-31
D 31-33
D 33-35
D 35-37
D 37-39
D 39-41
D 41-43
D 43-45
D 45-47
D 47-49
D 49-51
D 51-53
D 53-55
D 55-57
D 75-77
D 77-79
D 79-81
D 81-83
D 83-85
D 85-87
D 87-89
D 89-91
D 91-93
D 93-95
D 95-97
D 97-99
D 99-101
D 101-103
D 103-105
D 105-107
D 107-109
D 109-111
D 111-113
D 113-115
D 115-117
D 117-119
D 119-121
D 121-123
+1.000
-1.000
+1.000
-1.000
DIAGONALS.

FIG. 16.

rocker arm stress was entirely similar, but was here based on unity pull on the right rocker arm and $\frac{(P_L)_R}{(P_R)_R}$ on the left rocker arm.

This, in brief, covers the work necessary for the full determination of the replacing forces or the "indeterminate elements," as they are somewhat contradictorily called. From this point any usual methods of calculation become applicable, but the writer desires to insist on the advantages of continuing the work of actual calculation by the use of influence lines, as was done in the two cases here described. There are also certain important, although simple, modifications of the influence line method, here used, which are worth a little further description and the serious consideration of all who may ever undertake such work.

It may be assumed, of course, that all are familiar with the nature of influence lines and their use, but, for completeness, we may here describe an influence line as one, plotted from a given base line, which corresponds to the load line of the structure, with ordinates at each point of this base line equal (or proportional) in value to the influence of a load of unity at the corresponding point of the load line of the structure, on the part or member to which the influence line refers. The "influence" in question is ordinarily the stress at the part or in the member concerned, but it might as well be a strain or a displacement.

It will be assumed here that the reader is familiar with the methods of drawing and checking normal influence lines of statically determined structures, which are ordinarily composed of not more than three or four straight lines. We will therefore consider only influence lines for statically indeterminate structures.

The normal influence line of a statically indeterminate structure, often referred to as a curve because it is made up, at most, of straight lines, from panel point to panel point only, may be drawn by constructing first the simple influence line of the part concerned for the statically determined structure formed when the superfluous restraints are supposed removed and the superfluous bars cut. To each ordinate of this line may then be added (algebraically) the influences of the replacing forces on the part concerned, for load unity at the point of the structure corresponding to that of the influence line for which the ordinate is being determined. For this purpose we should first

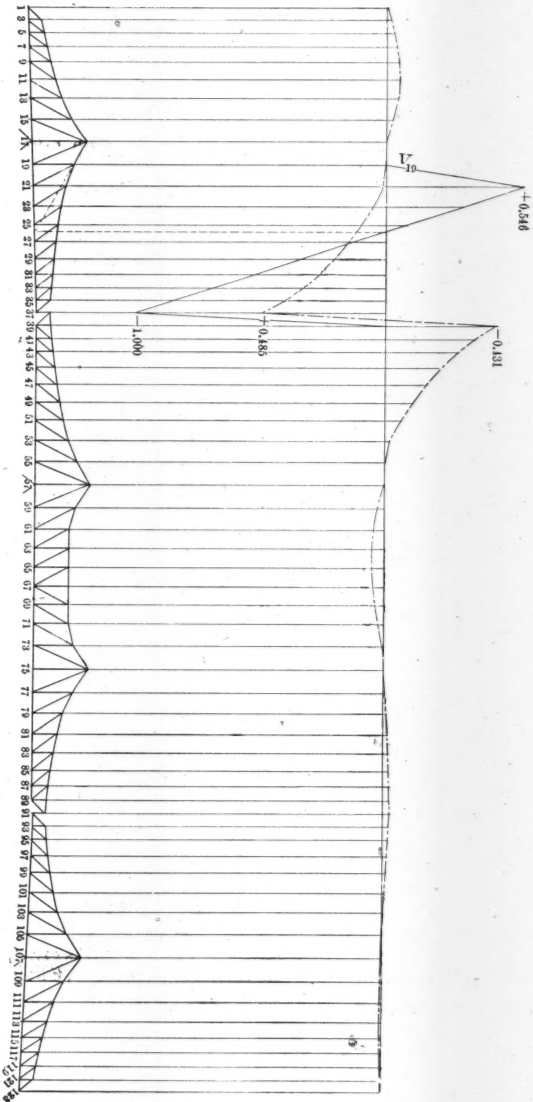


Fig. 17.

prepare an influence line of each of the replacing forces, as previously explained, and a diagram or table for each of the replacing forces, giving the "influence" of the replacing force, when of unity value, on each part of the reduced statically determined structure. These diagrams or tables would give the values of the stresses, S', previously mentioned, and so would already be at hand. The figuring of the "replacing forces corrections" of the statically determined influence line of a given part, then, simply involves multiplying the ordinate of the influence line of each replacing force at the load point for which we are determining the ordinate, by the value of the "influence" of that replacing force on the part whose influence line we are constructing, as shown by the diagram or table, and summing, algebraically, the corrections thus found for all the replacing forces. Nothing further is essential in this connection; but we may note an important variation on the ordinary influence line, which it is often or usually desirable to introduce. Instead of making up the ordinates of the final influence line by adding (algebraically) these replacing force corrections to the ordinates of the "statically determined influence line," we may lay off these corrections from the base line in the opposite sense, so that the ordinates between the line thus obtained and the statically determined influence line will have the final or total values. The correction line thus becomes, practically, the zero or base line.

When we have several replacing forces all influencing a given part, this is about as far as we can go in simplifying the work of constructing the influence lines. The use of these lines (when once constructed) gives at once the kind and amount of the influence of a load of unity at any point of the load line, and so permits us to determine at once where and how to place the loads for influence of a given sign (in particular, what loading to use for maxima and minima), and, then, to determine, by simple multiplication of each load into its corresponding influence line ordinate, and, by algebraic summation of the products, the consequent total influence. But in many, if not most cases, we have only one replacing force and, therefore, only one replacing force correction concerned with the part in question. Here a more brief and simple procedure becomes applicable, which may be called the method of distorted influence lines. If, under these circumstances, we proceeded in the ordinary manner, but plotted the

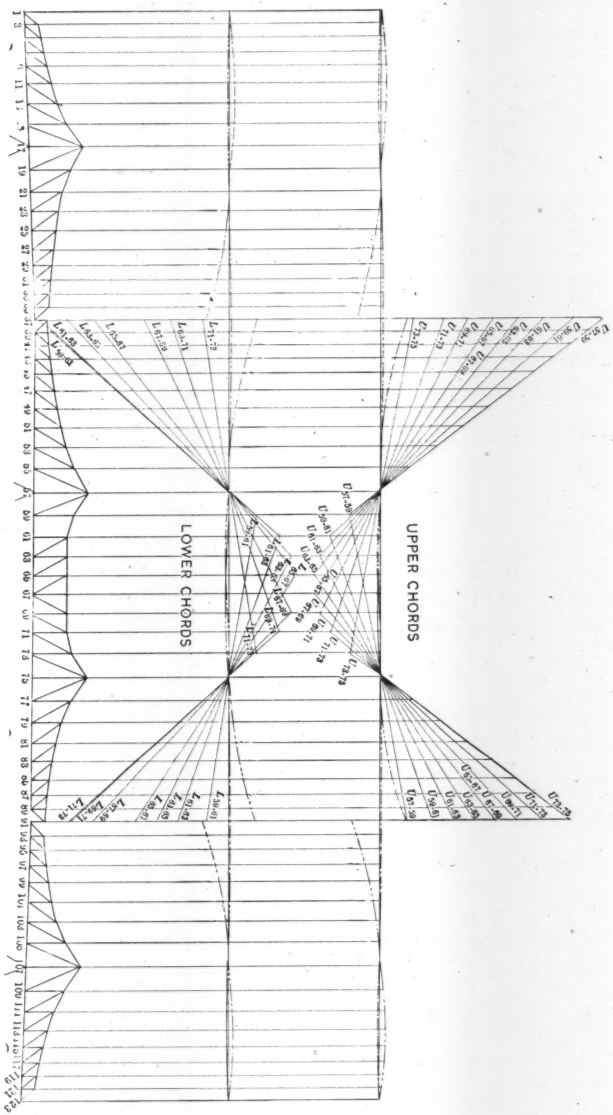


FIG. 18.

replacing force correction line as a zero or base line, as already described, this correction line would differ from the influence line of the replacing force only in having its ordinates different in a fixed proportion, that of the "influence" of unity value of the replacing force, on the part in question, or the influence coefficient. Instead, then, of calculating the ordinates of and plotting the replacing force correction line, we may more easily reduce the statically determined influence line by dividing its ordinates by the influence coefficient and plotting it in connection with the replacing force influence line. Then it combines with that influence line exactly as the normal statically determined influence line and the replacing force correction line would combine to give the true influence ordinates, except that the ordinates here are all reduced by division by the influence coefficient. But points of zero influence are given precisely as before, and the use of the distorted influence line is precisely the same and yields precisely the same results as does the use of the normal line, except that all ordinates and, therefore, all areas must be multiplied by the influence coefficient to give the true results.

But what is the advantage of this procedure? First, if there were but the single part we are considering, to which to apply the method (since we already have the influence line of the replacing force, and the statically determined influence line of our part is comparatively very simple), it is much easier to reduce the statically determined influence line of our part to, and combine it with, the replacing force influence line than to follow the reverse procedure. But, generally, the part is but one of many, all requiring similar treatment, and, since all of them can have their distorted statically determined influence lines plotted in combination with the one replacing force influence line, the advantage rapidly increases in proportion to the number of parts calling for treatment. And the saving of time and labor shows, not alone in plotting the influence lines, but even more in their subsequent use, in the determination of areas. The saving in the cases of the Manhattan and the Blackwell's Island Bridges from the use of distorted, instead of normal, influence lines was enormous. And the complication thus introduced was insignificant.

Consider our two illustrations. In the case of the Manhattan Bridge the replacing force was the stress in the cut anchor chain, or,

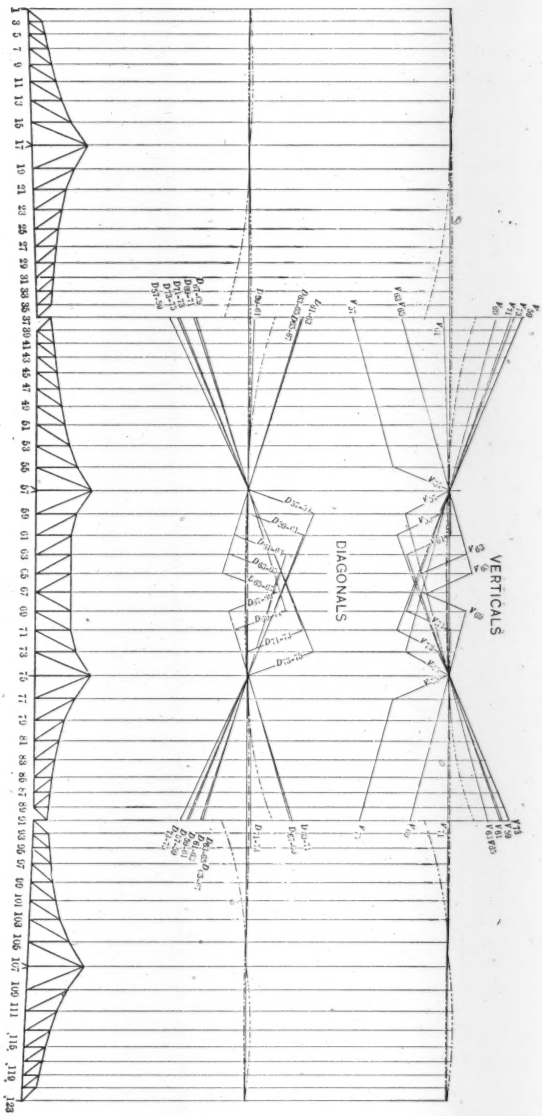


Fig. 19.

rather, its horizontal component. The influence line for this (or the *H* curve as it is called) was determined once for all and plotted but four times—on the sheet of top chord influence lines, on that of bottom chords, on that of verticals, and, finally, on that of diagonals. Then the distorted statically determined influence lines were drawn. Those for the bottom chords were exceedingly simple and regular. Those for the other members presented no real difficulties. The appearance of the four resulting sheets of influence lines can be seen from Figs. 3, 4, 5 and 6. One other feature of this case may be noted. The *H* line for a three-hinged design, with center hinge at the top chord, was also drawn, making all influence lines, with respect to it, influence lines for a three-hinged design. All the distorted statically determined influence lines were tested, first by the usual tests, on the supposition of three simple spans. They were then tested in connection with the *H* line for the three-hinged design. Finally, all calculations were made first for the three-hinged design, and the final corrections were those for the center bottom chord stress as a replacing force, instead of the anchor-chain horizontal component. This made the indeterminate corrections much smaller, and therefore easier to determine for the same degree of accuracy. It also furnished a set of maxima and minima stresses for the three-hinged design, as well as for the two-hinged design, the comparison of which may be made from the stress sheets for loading of 1 000 lb. per ft. given in Figs. 9 and 11.

In the case of the Blackwell's Island Bridge the replacing forces were the stresses in the so-called "rocker arms." Except for members of the center span of the Blackwell's Island truss, the stresses in the members were made up of a statically determined component and but one rocker arm correction. The entire Manhattan truss and the Manhattan cantilever of the Blackwell's Island truss involved only the left rocker arm stress correction. The influence line for the left rocker arm (indicated by a dash and one dot) having been found, all statically determined influence lines of these portions were distorted to combine with this rocker arm influence line, and the results are seen (some details excepted) in the left-hand portions of the distorted influence line sheets (Figs. 13, 15 and 16). The right-hand portions show the corresponding distorted influence lines for the Queens truss and the Queens cantilever of the Blackwell's Island truss.

based upon the right rocker arm stress (whose influence line is indicated by a dash and two dots). The distorted influence line of V_{19} is enlarged and shown separately in Fig. 17. The center span of the Blackwell's Island truss, involving a correction to the statically determined component from each rocker arm, requires a special rocker arm correction curve to be calculated for each member, to give the true influence for each load point. But this was not actually necessary in the calculations. Normal influence lines were used here (see Figs. 18 and 19).

This completes, in brief form, the description of the essential features of the methods used in making the check stress calculations. Much has been omitted which would have been pertinent and interesting were this a description of the actual work, instead of the methods, simply,—as, for instance, the treatment of certain special cases, many labor-saving devices, the nature of the checks introduced to guard against error and verify the conclusions, and, finally, the numerical values themselves and all that might result from their discussion. But all this would have exceeded the original scope of this paper and only distracted attention from its ends. Beyond mentioning that all the results were checked by the writer by independent methods and calculations which gave exceedingly close agreements, the writer considers it desirable to reserve all such matter for some future occasion.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 986.

ADDRESS AT THE THIRTY-SIXTH ANNUAL CONVENTION,

HELD IN THE

HALL OF CONGRESSES, ADMINISTRATION BUILDING, ST. LOUIS, MO.,

October 3d, 1904.

By Charles Hermany, President, Am. Soc. C. E.

1.—The Annual Convention of the American Society of Civil Engineers is, under provision of the Constitution, to be opened by an address from its President.

The great number of important papers which are to be presented to the International Engineering Congress has caused the customary annual retrospection of Engineering to be omitted in this address and the substitution therefor of a brief retrospection of the Profession in general, as an appropriate proceeding in opening this Convention.

2.—In January, 1904, when the speaker was elected President, the question as to the American Society of Civil Engineers joining in the enterprise of a Union Engineering Building in the City of New York was one of absorbing interest among the members. It was discussed with an interest, earnestness and thoroughness greater than given to

any other question that was ever before the Society. The engineering journals and the public press joined in the discussion to an unprecedented extent in a purely professional policy measure.

3.—In the discussion, the limitation and comprehensiveness of definition pertaining to the profession of Engineering were variously and conflictingly indulged in, but with benefit to the several distinctive branches thereof. The title of Civil Engineer was scrutinized with more than search-light penetration, as also with an apparent logical exactitude. The involution of the profession and its differentiations into many notable specialties were recognized, but there was great variance of opinion expressed upon the question as to whether the specialties were evolutions from the parent society into separate organizations, or whether they were still in the family and under its fostering care and protection. It is believed that the decision arrived at by the Civil Engineers was wise, and that it is generally acquiesced in by the members, though by some the opinion seems to be entertained that, through polytechnic school curricula and the constant broadening of the field of engineering, differentiations will continue which will result in such classification as will ultimately approximate a standardization of professional work.

4.—At that time the resolution was made to attempt, in this address, a review of the fundamental principles which underlie the profession, what relation it bears to the body politic, and what are the endowments, intellectual culture and accomplished work, upon which its claims as a distinctive modern profession are based, as deduced from the reading, observation, experience and reflection of one member.

5.—The American Encyclopædic Dictionary defines Engineer as "A person of genius or ingenuity." This definition names the primal personal qualities of an Engineer; it does not imply a profession, but may be regarded as the root thought from which has grown the generic term Engineer. If "A person of genius or ingenuity" is the lowest rung in the professional ladder, no one has yet, from the present top rung, traced him historically down, rung after rung, to the lowest implied in this definition. The profession of Architecture is historically more fortunate in this respect, for it can be traced back through the ages into Greek history, where we find "*Archi*," chief, and "*tekton*," carpenter, or "Chief Carpenter" as the origin of the present title

Architect. From this first distinct type, up to the present Architect, through all the art and science involved in his development, there is not found a broken link in the chain. Such connected or successive development cannot, at present, be intelligibly predicated in support of the Engineer. His involution seems to be the product of many irregular or variable factors, the connection between which seems vague and undefined.

6.—In the summer of 1854, fifty years ago this past summer, there was in the city of Cleveland, Ohio, a newly constructed office building, the rooms on the second floor of which were occupied, as offices, by doctors, lawyers, architects and engineers. With one of the latter I was then connected. Adjoining our office was a suite of rooms occupied by a firm of lawyers, with which there was connected a college graduate serving in the capacity of clerk, who, at convenient times, read law. There sprang up an intimacy between us through mutual calls upon each other, during which interchange of neighborly courtesies we discussed various matters, including the professions of law and engineering. Whenever engineering was the topic, the law student evinced much interest, but he seemed at times apparently disturbed at the thought that Engineers should presume to occupy quarters on the same floor and next door to those occupied by those of the learned professions. But he was civil, though not an engineer, and in part succeeded in subordinating his perturbation to the proprieties of neighborliness and gentlemanly deportment. After a lapse of several months, during which our relations were most friendly, he one evening seemed to be unusually interested in the profession of Engineering and asked questions most difficult to answer. I responded to the best of my ability, and talked, perhaps, more freely than wisely, when, apparently in great relief, he broke forth, by sententiously saying, "An Engineer is, then, after all is said and done, nothing but an educated mechanic." I understood his remark as a trifle dogmatic, or as coming from a man who had hitherto suppressed the feeling which afflicted the Pharisee who is reported to have prayed in the Temple, but this did not develop in me a publican on the steps; there was no contrition in, nor smiting of, my breast. I thanked him for the qualifying adjective, "educated," and added that the same rule of definition applied to the lawyer would make him only a pundit in the profession of law. Our roads had forked, and then, although it was

an amicable grade-crossing, the law of gravitation asserted itself audibly in our future professional transits. This law student's definition of Engineering, though not wholly incorrect, had a narrow range, although he was a man of high endowments and notable scholarly attainments. He was the son of the Reverend Charles G. Finney, then of Oberlin, Ohio.

7.—Robert Rawlinson's definition, given more than fifty years ago, was as follows: "To study precedent, to learn from experience and to correct from practice, are the chief duties of an Engineer," the correctness of which cannot be seriously questioned, but to contend that it is, at the present day, applicable to the profession at large, rather than to members of attainments in limited fields of practice, would be claiming too much for it.

8.—John Millington, at one time Professor of Mechanics and Civil Engineering in the Royal Institution of Great Britain, afterward Engineer in charge of a silver mining company in the State of Mexico, and later Professor of Chemistry, Natural Philosophy and Civil Engineering in William and Mary College, Va., published in 1839,

"ELEMENTS
OF
CIVIL ENGINEERING,
BEING AN ATTEMPT TO CONSOLIDATE THE PRINCIPLES
OF THE VARIOUS OPERATIONS
OF THE
CIVIL ENGINEER INTO ONE POINT OF VIEW."

However ambitious and praiseworthy such an effort may be considered to have been, it was, nevertheless, a Utopia never to be realized.

This manual was an octavo volume comprising 716 pages. It was largely devoted to definitions, and (excepting Professor Mahan's Elementary course of Civil Engineering) was the first notable contribution to engineering literature in the United States of America. The student of engineering may, even at this day, read it with both interest and instruction.

9.—The Encyclopedia Britannica (Ninth Edition) devotes a single paragraph of thirteen short lines to its definition of Engineering. This great repository of learning may, therefore, from this viewpoint,

be charged with having slighted a great profession with a modicum of recognition in the unlucky number, thirteen. In a compendium of such pretended profundity, this shortcoming is apparently an inexplicable blemish, though written in a language in which engineering literature is as extensive and rich as in any modern tongue.

10.—The Charter of the English Institution of Civil Engineers defines Civil Engineering as "The art of directing the great sources of power in Nature for the use and convenience of man." If this were written "To use the sources of power in Nature," it would include all, great and small. To substitute the verb, use, for the verb, direct, would be nearer correct. Direct implies authority; use, permission. Direct seems too broad, for the sources of power in Nature cannot be directed by man; they can be availed of, drawn upon, made use of, for transformation and service to man, but as sources they are so established in Nature as to be not subject to direction by man. In a physical or material sense, the Engineer's sources of power in Nature are apparently limited to Animals, Gravity, Heat, Magnetism and Electricity; the minor sources seem to be but transmutations of these, the primal sources, at least to the extent to which the present-day Engineer is concerned, or, in his limited capacity, can avail of them.

This view of the English definition seems to be warranted by the citation of the following well-known phenomena:

First.—By mechanism we cause gravity to vibrate a pendulum at intervals of a second of time in a clock, using the force of gravity to measure time, in primal units.

Second.—By mechanism we transform heat into mechanical energy, the last manifestation in one case at least is, weight and velocity, as in the fall of a pile-hammer, a transformation of heat into gravity.

Third.—By a simple mechanism, the magnetic needle guides the mariner across the uncharted oceans in comparative security.

Fourth.—By a train of mechanism, the last member of which is a carbon loop, we generate electricity and transform it into heat and light.

But in all these transformations we do not direct the source of gravity, the source of heat, the source of magnetism, or the source of electricity; we simply make use of them for specific purposes, without directing their sources.

11.—The Constitution of the American Society of Civil Engineers requires in candidates for membership, "The ability to design as well as direct engineering works," which definition is simple, broad and comprehensive. "Design and direct" is the essence of simplicity, yet sufficient to comprise everything needful to conceive, construct and complete an engineering work. It opens a field of absolute freedom to enter, unhampered by academic embellishments or dogmatic restrictions. The man who enters this field must do so prepared to contend for the prize of prominence, by making use of all the sources of power in Nature which are requisite to achieve the success which Engineering implies and involves.

12.—To recapitulate these seven definitions in the English language, it is deemed proper to say of them that:

The American Cyclopædic Dictionary's definition is the radical concept of the Engineer's personal bias or quality.

The law student's comprehends the subject only in part.

Robert Rawlinson's is too circumscribed.

John Millington's Utopia is unattainable.

The Britannica's definition is too limited and illiberal.

That of the English Institution goes to the heart of the matter, but is partly erroneous.

The American Society's is too broad and simple for adverse criticism.

13.—With this summary and review of definitions, one is led to an enumeration, analysis and classification of the powers in Nature which are perhaps the special heritage of the members of our profession. These sources of power are here enumerated in the historical order in which the Engineer has drawn upon them for the use and convenience of man.

The first source of power is the will and intellect of man made manifest through his anatomy; the second is the will, intelligence and docility of the trained domestic animals, displayed through their anatomies; the third is gravitation, the inexhaustible repository of ever-acting and imperishable force; the fourth is heat, with its endless differentiations; the fifth is magnetism, that early experienced but least definable source of power in Nature; and the sixth is electricity, with its wonderful and innumerable adaptations. These six may be regarded as the primal sources of power in Nature at present appre-

hended and drawn upon by the Engineer, of which sources all forms of power are but transmutations produced by the interaction of the principal sources and the agency of man.

14.—The before-mentioned seven definitions and six sources of power, taken collectively, may be assumed as comprising the, at the present day, scope of engineering, expressed in its achievement and literature. To announce or phrase all this in a simple sentence were to exhaust the resources of any language, or to symbolize it by an ideal figure, as is Justice by Libra in the hands of a blindfolded Goddess, were a desideratum devoutly to be wished, one that would confer immeasurable benefit upon the profession, for correct apprehension of principles and their clear annunciation form the starting point, whence alone success is attainable.

15.—The primal subdivisions of an Engineer's work consist of mental concepts of enterprises the prosecution of which necessitates the employment of many agencies, animate and inanimate, which must operate contemporaneously, if not, in a measure, simultaneously.

To marshal these agencies into a co-operative, active, artificial organism, requires three fundamental qualifications:

The study and mastership of mechanics as the science that constantly and imperatively guides and controls him in his successful enterprises.

The acquirement of the art of mechanism, without which none of his enterprises can come to successful issue, however well-grounded in the science of mechanics.

The possession of administrative capacity, which combines mechanics, mechanism and government, which make the man of affairs. Without this latter accomplishment, many otherwise capable Engineers fail in scoring the success due to their strictly professional attainments.

16.—A given state of civilization among a people makes desirable and possible great enterprises for promoting and continuing an uninterrupted development of such civilization, the primal law of which is progress or involution. The Engineer's province in this work is concerned largely with the physics which enter into the problem. His philosophy of civilization must be such as to have taught him correctly what is needful and possible. In engaging in any one of its

enterprises his conceptions must be correct, minute and comprehensive; he must be abreast or ahead of the times in which his enterprise is to materialize. Such conception means concentration and abstraction of mental processes of the utmost tense, compared with which diligence is as the temperature of the dewpoint is to solar heat. It constitutes an incandescence of vitality that implies the will, the intellect and the anatomy of a robust manhood. This accomplishment proceeds largely from heredity or natural endowment and is supplemented by training and culture.

17.—Mechanics is the law in accordance with which an engineering concept must be formulated, in order that it may constitute a physically logical scheme—a practicable enterprise; it is, in brief, the law according to which forces in Nature must be used in the province of Engineering. In simple terms, the science of mechanics is the philosophy of Engineering, and to the extent of his mastership in this science, the Engineer is qualified to materialize his concepts. It teaches him the scope and limits of his enterprises and how to discriminate correctly between the possible and the impossible. It constitutes a court of final resort in which his causes are physically adjudicated, whence no appeal is ever permitted and where immutable law dominates. In its dominion he is ever a subject, but never a sovereign, though he were a Hercules or a Demi-god. In the study and practicable application of mechanics, mathematics is the philosopher's stone, which enables the Engineer to transmute force, not into gold, but into the service of man, which is better than gold.

18.—Mechanism intervenes between all the uses of power and their results. As an art, it ranks second to mechanics as a science. The former is the result of man's efforts, while the laws of the latter are fixed in Nature; the former depends upon man, the laws of the latter are independent of man. The manifest differentiations of Nature undoubtedly suggested to man Mechanism, yet Nature provides nothing that takes the place of, or supplants, man's mechanism in engineering. Mechanism is ever mutable while Mechanics is ever immutable, yet their interrelation is so constant as to make analysis well nigh impossible. In man's development, Mechanism antedates Mechanics, but through the former, as an art, he rose to the apprehension of the latter, as a science. Mechanism is not self-existent, but has its scope limited only by man's ability to give it expression in the combination

of material parts. The scope and limits of Mechanics exist independently of the agency of man; limited transmutation and application is all that is possible to him.

19.—One of the triumphs in art, to which Mechanism lays indisputable claim, was the construction of pendulum clocks which kept correct time, a hundred years before the genius of Newton discovered, announced and formulated the law of gravitation. The interrelation of Mechanics and Mechanism seems to illustrate that existing between gravitation and motion—the one, cause; the other, effect. From the viewpoint of an Engineer, Mechanics appears to be a colossus in the realm of law, the anatomy and functions of which do not defy enumeration and analysis by him, but the marvels of Mechanism, his own work, in their endless differentiations, make complete enumeration and analysis impossible. These three primal subdivisions—concept, mechanics and mechanism—comprise the field of the civil engineer's activities. In the first, he stands alone; in the second, he enjoys the heritage of Nature and his predecessors; in the third, he is also an heir, but must design and produce additions of both novelty and usefulness, as the ever-recurring emergencies of his profession demand.

20.—Two hundred years ago civil engineering as a distinct profession had no existence. Not even claim was made for it in any language or literature, though irrigation in Egypt, India and other countries had been practiced for centuries. The Netherlands had been reclaimed from the sea. The bogs of the British Isles had been largely drained and transformed into arable land. Classic architecture had attained its utmost perfection and declined—to all appearances never to be revived. Great aqueducts, canals, roads and bridges had been built. All this had been done for the advancement of former civilizations by men who had the same endowments and much of the training of the modern civil engineer, though the materials they used were principally earth, wood and stone. Their sources of power were men and animals for all differentiating efforts. For stability in their structures, the force of gravity, through compressive stresses, was almost exclusively relied upon.

These men, moreover, were employed principally by State and Church, in populous and wealthy cities, to build monumental structures to commemorate the achievements of warlike nations and to symbolize human aspiration for a millennium religiously believed in

and universally sought. If these historic works, instead of belonging to the past, were to be in the future, they would be placed in charge of the Civil Engineer. History does not satisfactorily inform us what relation the men who "Designed and directed" these great works bore to the body politic. Their names are almost wholly forgotten, and those of the Kings and Emperors who assigned them to duty alone have been preserved. These men of genius and ingenuity seem to have been ranked but little higher than the citizens of their times, whom they directed in the enterprises in question. By their works alone we can know them, not through their personalities. What, then, is he, whom in modern times we call a Civil Engineer? In general terms, we may answer, the Civil Engineer in part transforms Nature for the material advancement of man. Wealth, as capital or conserved energy, has called him and he has responded, which wealth is apparently not sovereign, but subject, ever subject to the law according to and in conformity with which it is accumulated, which law is, tendency to endless activity and acquisition of conserved energy. This was true in former times, as it is true in modern times. In former ages the people had their energies and wealth forcibly appropriated by governments and dissipated in imperial enterprises for apparently no other purposes than the glory and benefit of the few, with degradation and injury to the many. Empires rose and fell like the flow of vapor, fire and lava from a volcano, with apparently no more regard for the welfare of the people than has the force of gravity for the placidity of the mobile ocean upon which, in sport, it plays an endless see-saw. In modern times this is no longer so generally and deplorably true, for now the energies and wealth of peoples are more largely directed, through agriculture, commerce and manufactures, for the benefit of all. The wisdom of conservation has been substituted for the folly of destruction. An era has dawned in which peace, instead of war, tends to pervade more largely the civilization of man, in which dawn the Civil Engineer has been differentiated from the Military Engineer, in which differentiation the adjective, Civil, has been applied with remarkable fitness. In this era, States of great intelligence, wisdom, wealth and power are built up, aggregated into nationalities, whose governments, though in rivalry with those of others, strive, as governments never strove before, to grow great and powerful, not to sub-

jugate and destroy others, but to elevate the citizen and cause him to participate more largely in this greatness and power than in former eras. Out of all this has been evolved the Civil Engineer, who finds here his habitat, his work and his glory. The modern Civil Engineer is not—like his predecessor, the Architect—principally employed by State and Church, to create monumental and symbolic structures in populous cities whose wealth is expended to commemorate the glory of empire and to give expression to unattainable aspirations in dreamers. His work is mainly a differentiation from the former ideals of man, in that he makes populous cities healthier and stronger, minimizes distances between them, brings them in daily connection in the affairs of life, such as commerce, travel, thought and speech, all of which tends more to the establishment of the actual brotherhood of man than has been accomplished through the ages preceding his advent.

This genesis of the profession is only a partial synopsis; volumes might be written upon it and not exhaust the subject. All this is, of course, a matter of common knowledge among Engineers. It is, upon this occasion, presented in condensed form as evidence that the Civil Engineer has demonstrated his rank to be that of a professional man in the body politic; not only a professional man, but one who ranks high, with an evident tendency to outrank some of the learned professions of to-day, because he makes individual and collective man greater and stronger, instead of causing him to ever ponder his weakness and dependence, as some professions do. He knows of no restrictions or limitations, but dares all within Nature's and his own resources. He speaks, not in a florid, sonorous and profound philology, but in bold constructions, a language in visible material realities which are legible by, and address themselves to, the masses, as making for their advancement and benefit. The ideals of the modern Engineer are given expression in the construction of artificial and the improvement of natural waterways; in the building, equipping and operating of railways; in the construction of power-stations that marshal and direct forces for the service and preservation of man, which are incomparably greater than were the forces of Xerxes's army for the overthrow of the Greeks and the destruction of their Empire; and in the construction of manufacturing establishments for better protecting, housing, feeding and clothing the people than previous ages could lay claim to or boast

of. The Erie Canal was for a long time the ideal of Engineers, and when, eighty-seven years ago, its construction was commenced, the Engineers' available materials were earth, stone, wood and iron, with man, animals, gravity and heat as sources of power. The Niagara power development was also an ideal of Engineers, and when its construction commenced, seventy-five years later, the materials available had been many times multiplied with all the present-day known sources of power in Nature at command. The ideals given expression in these completed enterprises explain, in part, what the modern Civil Engineer is, and upon what, in part, he bases his claim of being a professional man.

21.—To the layman and the public in general, all that which is here expressed and implied may seem but the common every-day fruitage that accompanies agriculture, commerce and manufactures; that this is but an expression of our age's stupendous materialism; that it has not produced any literature, except a technique in occult phrasing, the comprehension of which is only interesting or possible to those who lead a life of monkish devotion to a calling. To the Engineer, there is perhaps not more, if as much, meat in all this, than to the public, but above and beyond this material comprehension, there is an undefinable something that sustains and buoys the Engineer with a daily inspiration, as though he were a veritable "Aurora" unembarrassed by mythological sex classification.

22.—There is a sentiment prevalent among Engineers that Art must not be permitted to form a prominent feature in engineering works; that it tends to no useful end, involves needless cost, and that it rather impairs use, the sole object in engineering enterprises. This is but a half truth, which, in Engineering, as in all the affairs of man, ranks equal in mischievousness with error or untruth.

The hand-hammer's type, suggested by the human fist and arm, was originally only a club, picked up by man in the forest, which, by use and time, became undoubtedly the first tool by which muscular force was made more effective through velocity and weight, a distinct elementary power implement. Archæology teaches us that this tool has undergone many transformations, until, at the present day, it is as indispensable as any, and in such variety of forms and adaptation as to present forcibly the element of Art as an inseparable accompaniment of this first and simplest tool. It demonstrates, further,

that man is constrained, in his work of continuing any one operation, to invest it with an appropriate art form. In like manner, down the ages, in whatever man has done, artistic design, in form, proportion and volume, is found to be an inseparable and pronounced feature. Is it, then, possible to eliminate Art from engineering works? The answer is, "no." The work of the Engineer the world over bears irrefutable testimony against it. Those members of the profession who decry it fail to that extent in rising to the proper dignity which Engineering implies. The extraneous loading of engineering works with ornamentation and decoration is, of course, needless and vicious, for such additions are but questionable appendages to any form of art, but in the true art sense, engineering works rival all forms of art, though the modern engineer, when he presents to us ideals, idols or toys, erects Obelisks on Bunker Hill and the Potomac Flats, a Statue of Liberty in the harbor of New York, an Eiffel Tower on the Champs de Mars, and a Ferris Wheel on the shore of Lake Michigan. Every engineering structure is but a correct balancing of all the forces it comprises, and, in whatever material presented, is membered or articulated so that the trained eye reads the direction of the stresses involved, which is the highest possible expression of constructive art, partly because it is true and partly because its utility or serviceability to man is universally manifest. It symbolizes its purpose and expounds the motive and character of its author in unmistakable and irrefutable terms. It constitutes a voiceless herald that brings joy to the multitudes that gaze upon it, for they know that it is their redeemer from despairing drudgery and burdensome labor. This is not the case with what is called the fine arts, the meaning and service of which latter are perhaps supremely idealized, specialized and limited to the apprehension and appreciation of those who have what is called cultivated taste, while the benefit derived therefrom is an exalted and undefinable something which eludes description, even by the language of the poet. No attempt is here implied to depreciate high art, but to contend that engineering works are not destitute of true art, cannot be made to be so, but admit of, if they do not necessitate, art forms and proportions which fill the soul of man with awe and admiration.

In support of this claim, take, as evidence, a transcontinental train of express, mail, passenger and sleeping cars, drawn by a modern

locomotive, rolling over two slender ribbons of steel, through the mountain defiles of the East; over the plains of the prairie plateau; through populous cities, blooming gardens, teeming farms and arid, waterless deserts; up the cañons of the Rocky Mountains; over the backbone of the continent; down the Western slope of the Pacific; crossing, on the route, rivers on spider-web-like bridge structures; through dark tunnels, miles in length, regardless of peril and accident; through sunshine and rain; in storm and blizzard; distancing the flight of birds and rivaling the continental sweep and velocity of the cyclone; from the Atlantic to the Pacific Ocean; all accomplished with clock-work regularity in less time than a week; and you have an array of Art-accentuated panoramas which astound and charm you while they verify the claim. Were this but an ideality, it would command admiration; but when you realize it as a daily accomplished feat, you are contemplating events, compared with which the wonders of past ages are commonplace in their tendency to arouse the spirit of man in its onward and upward flight. This is principally the work of modern Engineering, and explains, in part, what the men are whom we rightly call Civil Engineers.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No 987.

SOME NOTES ON THE CREEPING OF RAILS.*

By Samuel Tobias Wagner, M. Am. Soc. C. E.

With Discussion by Messrs. G. Lindenthal, W. M. Camp, F. S. Stevens, Hunter McDonald, P. H. Dudley, George Tatnall, Joseph S. Ward, Charles Rufus Harte and Samuel Tobias Wagner.

In January, 1903, the writer commenced a series of observations, for the Philadelphia and Reading Railway Company, upon the creeping of rails at a number of points on the Company's lines. This work was undertaken in order that certain questions propounded by Dr. P. H. Dudley, Reporter for America, on "Ways and Works, Rails for Lines with Fast Trains," for the Fifth International Railway Congress, could be answered as intelligently as possible. The writer is enabled to present these data to the Society by the courtesy of Theodore Voorhees and William Hunter, Members, Am. Soc. C. E., with the hope that they will prove of interest and will elicit discussion on a very interesting subject to the railroad engineer.

Object of the Investigation.

Among the questions, in the circular letter of Dr. Dudley above referred to, were ten under the heading "Methods for preventing creeping, especially on double-track lines and on steep gradients,"

* Presented at the meeting of September 7th, 1904.

and the investigations were made because there were no records of rail movements from which even ordinary answers could be obtained. That there is a longitudinal movement of the rails, under certain conditions, is a firmly established fact, but how it was influenced by conditions of roadbed, weight of rail, grades, traffic, etc., remained to be determined. The results of the observations, as will be shown later, are too few to establish firmly any laws, but it is hoped that what has been done and is here presented will make one link in a chain which ultimately may allow certain definite principles to be laid down. The observations were not undertaken to ascertain the best method of preventing the creeping of rails, and therefore no discussion of this interesting feature is presented.

Method of making the Observations.

In order to make the measurements as carefully as possible, two stakes, 3 by $1\frac{1}{4}$ in., and 24 in. long, were driven level with the ground about 15 or 20 ft. from the nearest rail, where possible, and located at right angles to the track, if on a tangent, or on a radial line, if on a curve. They were placed so as to be approximately in the center of two rails and miss any joints. In the top of each stake an ordinary surveyor's tack, with a depressed center, was driven. A transit was set up over one stake and sighted on the other, and points on the line were marked on the flange of each rail with a fine center-punch. All observations were made on double-track lines, and, on the tops of the two outside rails, similar marks were made.

From these marks on the outside rails measurements of exactly 30 ft. were made to the right and left, spanning a joint in each direction, and marked with the same center-punch. These points were placed in order to try and determine whether there was any movement in the joints due to expansion or contraction, or whether, when there was a longitudinal movement, it was transmitted through the joints to the next rail. The division superintendents were requested to notify the repairmen to avoid these joints as much as possible during the continuance of the observations. It is safe to say that no disturbance of the track took place during the time the observations were being made, unless noted in the tables.

A card, similar to that shown in Fig. 1, was prepared for each rail. A center line was drawn across the card terminating at the top in a

regular-shaped V. The card was then placed on the flange of the rail, a sharp-pointed pencil was placed in the center-punch mark and the card oriented by the observer at the transit. The card was then placed firmly against the rail, the pencil removed, and a point marked on the card on the line given by the observer at the transit which had been previously set on one stake and sighted on the other. Upon returning to the office this point was transferred to the opposite side of the center line, as in its first position it indicated a reverse movement of the rail.

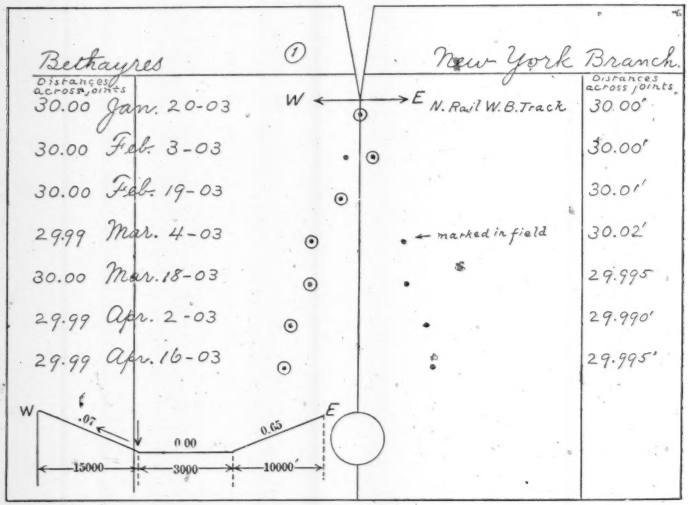


FIG. 1.

On the cards prepared for the outside rails, the distances across the joints were recorded, marked "*X*" when measured to the left, and "*Y*" when measured to the right. Temperature observations were also made and recorded, the thermometer having been exposed to the direct rays of the sun while the observations were being made. The length of time consumed in making the observations approximated about 15 min., depending upon the difficulty in finding the punch marks obscured by grease, dust, dirt, snow, ice, etc., on the rail. The work required the services of two men, one with the transit and the other handling the cards on the rails.

This method was decided upon after some discussion, and adopted

because it seemed to eliminate errors in making the notes, and at the same time showed on the cards the actual movements, without the necessity of measuring small distances, with the probabilities of errors in reading.

In most cases the observations were made at intervals of about two weeks, with the exceptions noted in the records. On the Atlantic City Railroad, the intervals were a month apart, on account of the extremely slow movement of the rails.

The center-punch marks on the tops of the rails on some of the branches had to be constantly remarked, especially where there was heavy traffic and heavy grades combined, as on the Frackville Branch, and on the Mahanoy and Shamokin Branch. Several observations were lost in this manner.

Description of the Points Selected.

The plan for making the observations contemplated the following outline, with special reference to grades and traffic:

1.—Lines having the greatest high speed, light tonnage and lightest grades;

2.—Lines having high speed, heavy tonnage and light grades;

3.—Lines having average speed, average tonnage and moderate grades;

4.—Lines having slow speed, heavy traffic and heavy grades.

In most cases inquiry was made before the points were selected, to ascertain where creeping had been noticed, with a general purpose of selecting the points so that they would fulfill most of the requirements.

The following are detailed and concise descriptions of the features of interest on the branch roads on which the points were selected.

Atlantic City Railroad.—This road is a double-track line, extending from Kaigns Point, Camden, N. J., to Atlantic City, N. J., and carries moderate traffic, compared with the other lines considered, but most of the passenger traffic is carried at high express speeds, the schedule of the so-called 60-min. trains being 50 min. from Camden to Atlantic City, a distance of 55.5 miles. The freight business is not heavy. The alignment throughout is very good, and the grades, with one exception, between Clementon and Albion, very light.

At the points marked Clementon and Albion both tracks are laid

with 90-lb. rails of the section recommended by the American Society of Civil Engineers, and have stone ballast.

At Pleasantville and Farmington the south-bound track was laid with 90-lb. rails and stone ballast and the north-bound track with 80-lb. rails and cinder ballast. At Pleasantville the point is on an embankment over the salt marsh of the Atlantic City meadows.

The point at Clementon is in a sump between two of the heaviest grades on the line. At Albion the point was selected as being at the bottom of the heaviest single grade.

On account of the high-speed traffic, the physical condition of the road is very good.

New York Branch.—This branch is a double- and in some cases a three-track line extending from Wayne Junction, Philadelphia, to Bound Brook, N. J., a distance of 54.9 miles. A very heavy traffic is carried, the passenger business being carried at high speeds. There are eight passenger trains daily in each direction, making the trip from Wayne Junction to Bound Brook in 58 min., while five other trains each way daily make the same run in 66 min. The line is equipped throughout with 90-lb. rails and stone ballast. The maximum grade is 0.7 per cent.

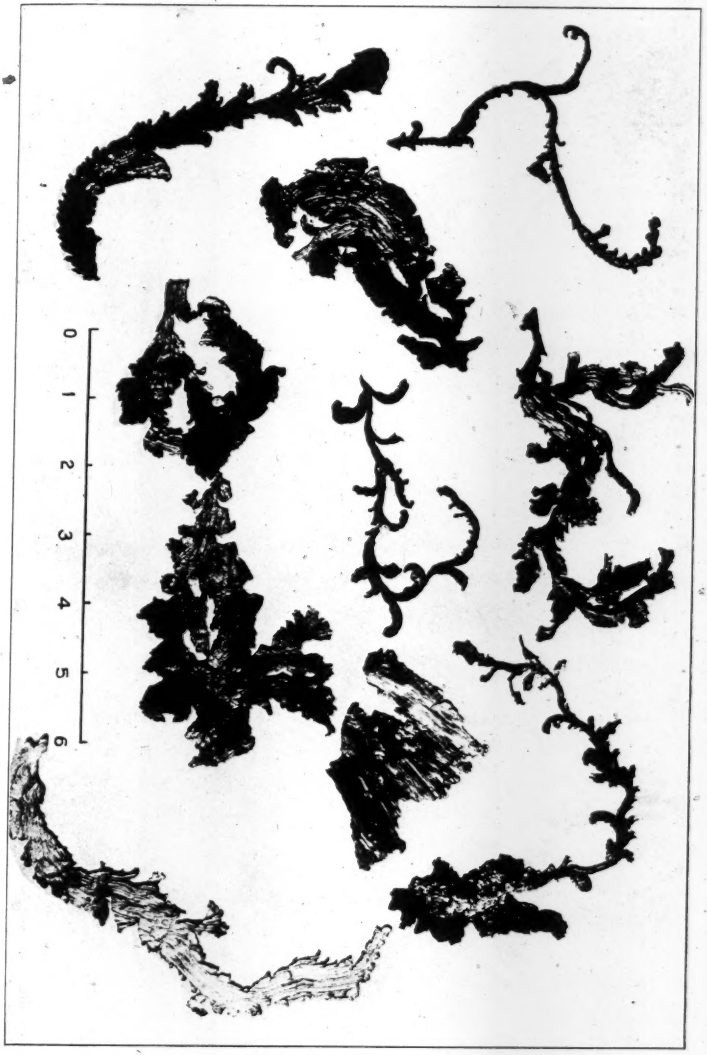
At Bethayres the point is located at the foot of a 0.7% grade which is about 15 000 ft. long, from the west. There is a level stretch of about 3 000 ft. and then a grade of 0.65%, about 10 000 ft. long, to the east. The point is on an embankment over low swampy ground.

Philmont is at the top of the 0.65% grade referred to at Bethayres, and is in a good dry cut.

Neshaminy Falls is at the foot of another 0.65% grade which runs out level. It is a point where the maximum speed is attained by express trains. It is on an embankment, but on a hard substratum.

Hopewell is in a cut on a rather soft substratum, and is approached from the west on a level tangent about 6 000 ft. long, and from the east at the foot of a 0.7% grade about 4 000 ft. long.

Bethlehem Branch.—This branch leaves the New York Branch at Jenkintown and extends to Bethlehem, Pa., a distance of 46.0 miles. It is double-tracked throughout. The grades are heavier than on the New York line, the maximum being 1.13 per cent. The traffic is not as heavy nor the speed as fast as on either of the lines already mentioned. As a general thing, also, the weight of the rail is 80 lb. Rail



FRAGMENTS OF STEEL FROM LOCOMOTIVE TIRES.
The scale on the photograph represents inches.

creeping was reported to have been noticed near a number of the points selected.

Quakertown is on a level stretch of track slightly beyond the foot of a 0.38% grade. The south-bound rail is 90-lb. and the north-bound 80-lb. The ballast is of furnace slag.

North Wales is on a 30-min. curve near the summit of a long 1.13% grade. It is in a rock cut. The track is laid with 80-lb. rails, and the ballast is furnace slag.

Gwynedd is on a tangent at the foot of a 1.13% grade, about 11 000 ft. long. The rails are 80-lb., and the ballast is furnace slag.

Oreland is near the foot of a 0.8% grade about 8 000 ft. long. The rails and ballast are the same as at Gwynedd.

Frackville Branch.—This branch extends from the top of the inclined plane at Frackville, Pa., to a point near Pottsville, Pa., and carries a very heavy south-bound traffic of loaded coal cars down a grade of 3.3% for a distance of 3.8 miles. The north-bound traffic is practically all empty coal cars. Nearly all the coal from the Shamokin Coal Fields, to tidewater at Philadelphia, is brought over the Mahanoy and Shamokin Branch to Mahanoy Plane. There it is pulled up an inclined plane with stationary engines to Frackville, whence it passes down the other side of the mountain, as described.

Head of Grade is at the top of the mountain, not far from the summit, and Broad Mountain is at the foot of the 3.3% grade.

The rails are 90-lb. on both tracks, and the ballast is of engine cinders. As there is absolutely no fast traffic, and only a few passenger trains each way daily, the condition of the roadbed is not kept up to the same standard as on the other branches already mentioned. The line formerly was laid with 80-lb. rails, the 90-lb. rails being put down only recently. Intermediate angle-splices were put on the 80-lb. rails at points on the south-bound track where the creeping was greatest, but, at the time the observations were made, they had not been put on the 90-lb. rails at the points of observation. They are now on. These are fastened to the rails with old splice-bars with two bolts, and spiked to the ties through the slots.

Mahanoy and Shamokin Branch.—This is part of the Shamokin Division, the points selected being on the south side of Locust Mountain, on the heaviest grades of the line. This is on the main line from Tamaqua to Shamokin, Pa. The grade is 2.5% and the traffic heavy,

but at moderate speeds on account of the grades. The heaviest traffic, which is made up largely of the coal from the Shamokin District, is on the south-bound track. On the south-bound track, 90-lb. rails are used, and 80-lb. on the north-bound. The ballast is of engine cinders.

Locust Summit is at the top of the grade, and is in a cut which is rather wet. Locust Dale Junction is at the foot of the same grade.

The wear of the rail is very marked on the south-bound track, on account of the heavy braking required on loaded trains. This braking is very hard on the brake-shoes and tires of the engines, as is shown by the pieces of steel removed from the engine tires, and found along the track. Photographs of some of these are shown in Plate XXIX. Similar pieces are found on the south-bound track of the Frackville Branch.

The Observations.

The results of the observations are shown in Tables 1 to 4, and in Plates XXX to XXXIII, in which are given all the conditions of the location and traffic, as well as the rail movements. The alignment, character of the roadbed, manner in which the joints are spiked to the ties, cut or fill, traffic, detailed location of the points selected, with reference to the joints and the grades, are given in order to aid in an intelligent study.

Abstract from Camp's "Notes on Track."

Probably the best general discussion of the subject of creeping rails is that given by W. M. Camp, M. Am. Soc. C. E., in his "Notes on Track," and the writer has taken the liberty of extracting from it a few notes on the general principles. These are given with the object of presenting concisely the best known data on the subject, so that the results here obtained may be better understood.

There are two longitudinal movements in rails:

1.—A molecular movement of expansion and contraction, and

2.—A progressive shifting of the rails bodily, known as creeping.

It is now generally conceded that the principal cause of creeping is the wave motion in the rails set up by moving trains.

If each rail, under this wave motion, is not made to hold fast to the ties and the ground, it will remain shoved ahead, by a very small amount, by each passage of a train.

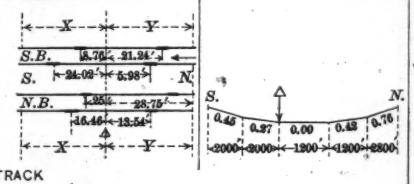
PLATE XXX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIII, No. 987.
WAGNER ON
CREEPING OF RAILS.

RECORD OF CREEPING RAILS. ATLANTIC CITY R.R. 1903.

Temperature taken in sun.

CLEMENTON

LOCATION OF JOINTS



GRADE

S.B.TRACK

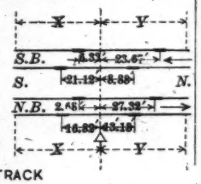
			X		Y
W. Rail	Jan. 14	29°	30.00	←	30.00
	Feb. 11	39°	29.99		"
	Mar. 11	50°	29.985		"
	Apr. 7	61°	29.98		"
	May 7	73°	29.96		"
	Nov. 24	46°	29.99		29.995
E. Rail	Jan. 14	29°		←	
	Feb. 11	39°			
	Mar. 11	50°			
	Apr. 7	61°			
	May 7	73°			
	Nov. 24	46°			

N.B.TRACK

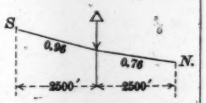
W. Rail	Jan. 14	29°		→	
	Feb. 11	39°			
	Mar. 11	50°			
	Apr. 7	61°			
	May 7	73°			
	Nov. 24	46°			
E. Rail	Jan. 14	29°	30.00	→	30.00
	Feb. 11	39°	"		"
	Mar. 11	50°	"		"
	Apr. 7	61°	29.985		"
	May 7	73°	29.975		"
	Nov. 24	46°	29.99		"

ALBION

LOCATION OF JOINTS



GRADE



S.B.TRACK

			X		Y
W. Rail	Jan. 14	30°	30.00	←	30.00
	Feb. 11	40°	"		"
	Mar. 11	56°	"		"
	Apr. 7	62°	"		"
	May 7	90°	"		"
	Nov. 24	44°	"		29.98
E. Rail	Jan. 14	30°		←	
	Feb. 11	40°			
	Mar. 11	56°			
	Apr. 7	62°			
	May 7	90°			
	Nov. 24	44°			

N.B.TRACK

W. Rail	Jan. 14	30°		→	
	Feb. 11	40°			
	Mar. 11	56°			
	Apr. 7	62°			
	May 7	90°			
	Nov. 24	44°			
E. Rail	Jan. 14	30°	30.00	→	30.00
	Feb. 11	40°	"		"
	Mar. 11	56°	29.99		30.01
	Apr. 7	62°	"		"
	May 7	90°	29.985		30.00
	Nov. 24	44°	29.98		"

FARMINGTON

LOCATION OF JOINTS

GRADE

N.B.TRACK

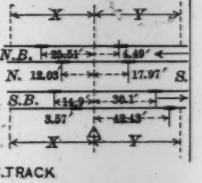
			X		Y
E. Rail	Jan. 15	43°	30.00	←	30.00
	Feb. 11	44°	"		"
	Mar. 11	58°	"		"
	Apr. 8	69°	"		"
	May 8	76°	"		"
	Nov. 10	55°	"		"
W. Rail	Jan. 15	43°		←	
	Feb. 11	44°			
	Mar. 11	58°			
	Apr. 8	69°			
	May 8	76°			
	Nov. 10	55°			

S.B.TRACK

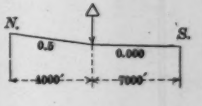
E. Rail	Jan. 15	43°		→	
	Feb. 11	44°			
	Mar. 11	58°			
	Apr. 8	69°			
	May 8	76°			
	Nov. 10	55°			
W. Rail	Jan. 15	43°	30.00	→	30.00
	Feb. 11	44°	"		"
	Mar. 11	58°	"		"
	Apr. 8	69°	"		29.995
	May 8	76°	29.99		29.98
	Nov. 10	55°	30.00		29.995

PLEASANTVILLE

LOCATION OF JOINTS



GRADE



N.B.TRACK

			X		Y
E. Rail	Jan. 15	43°	30.00	←	30.00
	Feb. 11	42°	"		"
	Mar. 11	55°	29.99		"
	Apr. 8	55°	"		"
	May 8	90°	29.985		"
	Nov. 10	60°	29.99		"
W. Rail	Jan. 15	43°		←	
	Feb. 11	42°			
	Mar. 11	55°			
	Apr. 8	55°			
	May 8	90°			
	Nov. 10	60°			

S.B.TRACK

E. Rail	Jan. 15	43°		→	
	Feb. 11	42°			
	Mar. 11	55°			
	Apr. 8	55°			
	May 8	90°			
	Nov. 10	56°			
W. Rail	Jan. 15	43°	30.00	→	30.00
	Feb. 11	42°	"		"
	Mar. 11	55°	"		"
	Apr. 8	55°	29.99		"
	May 8	90°	29.98		"
	Nov. 10	56°	29.99		"

The actual movements of rails are twice as great as those shown on this plate.

If it were possible to tighten the splices so as to hold against creeping, they would be too tight to allow the rails to expand easily, and much evil would result.

Three important facts should be noted:

1.—The creeping is most rapid during hot weather;

2.—It is greater on double- than on single- track; and

3.—It moves with the trains.

Rails usually creep most on embankments, especially on those newly made, and little or least on solid, hard ground, not raised above the surrounding level. Track laid on swampy or boggy land creeps worst of all.

The manner in which rails will creep, and the amount, depends on:

1.—The character of the ground or foundation for the track;

2.—The direction in which the train loads are heavier;

3.—The proportion of the weight distributed on the two rails;

4.—The speed of the trains; and

5.—The manner in which the ties are spiked.

What Seems to be Shown by the Observations.

A very brief summary of what seems to be shown by the observations is as follows:

1.—At 32 different points to determine which rail moved the more, the right or the left, the following was found:

At 21 points there was no practical difference.

At 8 points the right rail moved more.

At 3 points the left rail moved more. The right crank leads on all engines.

2.—In 7 cases out of 12 the greatest creeping is shown on descending gradients; on the remaining 5 there is practically no difference.

3.—Decidedly more creeping is shown where the roadbed is carried on embankment over swampy ground, than in other places.

4.—More creeping is generally shown on imperfectly maintained track, than where it is kept in first-class condition.

5.—As far as the observations went, it was shown that there was less movement than was expected from the reports of the repairmen, although, in some cases, the points were not at the exact locations of the worst reported creeping.

One of the most interesting points of the series is that at Bethayres, on the New York Branch. At the time the points were located new rails had just been laid, in cold weather, and it had been impossible to space the ties so as to spike all the joints to them. The result was a considerable amount of motion. Between March 17th and April 2d, the west-bound track was fixed, the ties respaced and all the spikes driven in the joints. In spite of this, the movement continued, and was probably due to the disturbance of the roadbed. The Philadelphia, Newtown and New York Railroad crosses this branch a short distance east of this point with a grade crossing, and when the east-bound track was fixed up, about May 2d, the rails were disturbed between the point of observation and the crossing, but the motion continued afterward. The excessive motion is probably due to the fact that at this point the track is not as rigid, longitudinally, as at other points, on account of the crossing frogs at the Newtown crossing and, also, chiefly on account of the swampy ground under the embankment.

The next most serious movement seems to be at Pleasantville, on the Atlantic City Railroad, over the salt marsh, where the movement has been continuous and regular on a nearly level line. The interval of time between the last two observations should be noted.

These two points seem to bear out the principle that creeping is most to be feared when the character of the foundation permits an amplification of the wave motion. The speed of all trains at both of these points is high.

There are a number of facts shown in the tables and diagrams which are very hard to explain, but it is hoped that they may throw some light on points where other records are wanting.

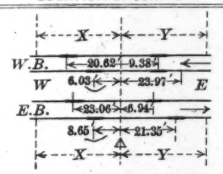
It is to be regretted that more observations were not made on curves, in order to ascertain the effect of the unequal distribution on the two rails caused by heavy, slow-moving freight trains passing around a curve having the proper superelevation for the higher speed of passenger trains.

RECORD OF CREEPING RAILS. NEW YORK BRANCH. P. & R. RY. 1903.

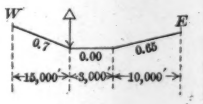
Temperaure take

BETHAYRES

LOCATION OF JOINTS



GRADE



W.B. TRACK

			X		Y
	Jan. 20	32°	30.00		30.00
	Feb. 3	49°	"		"
	Feb. 19	15°	"		30.01
	Mar. 4	59°	29.99		30.02
N. Rail	Mar. 17	61°	30.00	Ties replaced, respaced and	29.995
	Apr. 2	68°	29.99	spiked at joints.	29.99
	Apr. 16	49°	"		29.995
	May 2	75°	⊗ "		"
	May 13	90°	"		"
	May 29	90°	"		"

	Jan. 20	32°	
	Feb. 3	49°	
	Feb. 19	15°	
	Mar. 4	59°	
S. Rail	Mar. 17	61°	Ties replaced, respaced and
	Apr. 2	68°	spiked at joints.
	Apr. 16	49°	
	May 2	75° ⊗	
	May 13	90°	
	May 29	90°	

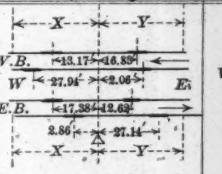
E.B TRACK

	Jan. 20	32°	
	Feb. 3	49°	
	Feb. 19	15°	
	Mar. 4	59°	
N. Rail	Mar. 17	61°	
	Apr. 2	68°	
	Apr. 16	49°	Replacing ties and spiking joints
	May 2	75° ⊗	
	May 13	90°	
	May 29	90°	

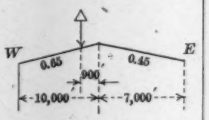
	Jan. 20	32°	30.00		30.00
	Feb. 3	49°	30.01		30.01
	Feb. 19	15°	30.03		30.012
	Mar. 4	59°	"		30.015
	Mar. 17	61°	"		30.01
S. Rail	Apr. 2	68°	"		30.025
	Apr. 16	49°	30.035	Replacing ties etc.	30.02
	May 2	75° ⊗	30.04		30.02
	May 13	90°	"		"
	May 29	90°	"		"

PHILMONT

LOCATION OF JOINTS



GRADE



W.B. TRACK

			X		Y
	Jan. 20	34°	30.00		30.00
	Feb. 3	52°	29.995		"
	Feb. 19	16°	30.00		"
	Mar. 4	59°	29.985		"
N. Rail	Mar. 17	71°	"		"
	Apr. 2	68°	29.98		"
	Apr. 16	56°	29.98		29.990
	May 2	73°	"		"
	May 13	101°	"		30.00
	May 29	94°	29.99		29.99

	Jan. 20	34°	
	Feb. 3	52°	
	Feb. 19	16°	
	Mar. 4	59°	
S. Rail	Mar. 17	71°	
	Apr. 2	68°	
	Apr. 16	56°	
	May 2	73°	
	May 13	101°	
	May 29	94°	

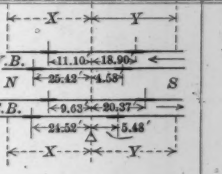
E.B. TRACK

	Jan. 20	34°	
	Feb. 3	52°	
	Feb. 19	16°	
	Mar. 4	59°	
N. Rail	Mar. 17	71°	
	Apr. 2	68°	
	Apr. 16	56°	
	May 2	73°	
	May 13	101°	
	May 29	94°	

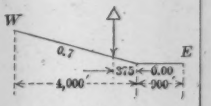
	Jan. 20	34°	30.00		30.00
	Feb. 3	52°	"		"
	Feb. 19	16°	"		30.03
	Mar. 4	59°	"		"
S. Rail	Mar. 17	71°	"		29.995
	Apr. 2	68°	"		30.00
	Apr. 16	56°	"		30.01
	May 2	73°	"		29.995
	May 13	101°	"		29.99
	May 29	94°	29.995		"

NESHAMINY FALLS

LOCATION OF JOINTS



GRADE



W.B. TRACK

			X		Y
	Jan. 20	40°	30.00		30.00
	Feb. 3	54°	"		"
	Feb. 19	18°	30.01		"
	Mar. 4	70°	29.99		"
N. Rail	Mar. 17	69°	"		"
	Apr. 2	72°	29.995		"
	Apr. 16	54°	29.99		30.01
	May 2	65°	"		"
	May 13	96°	29.995		29.995
	May 29	102°	29.99		30.00

	Jan. 20	40°	
	Feb. 3	54°	
	Feb. 19	18°	
	Mar. 4	70°	
S. Rail	Mar. 17	69°	
	Apr. 2	72°	
	Apr. 16	54°	
	May 2	65°	
	May 13	96°	
	May 29	102°	

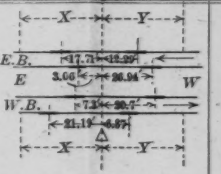
E.B. TRACK

	Jan. 20	40°	
	Feb. 3	54°	
	Feb. 19	18°	
	Mar. 4	70°	
N. Rail	Mar. 17	69°	
	Apr. 2	72°	
	Apr. 16	54°	
	May 2	65°	
	May 13	96°	
	May 29	102°	

	Jan. 20	40°	30.00		30.00
	Feb. 3	54°	"		"
	Feb. 19	18°	"		"
	Mar. 4	70°	"		"
S. Rail	Mar. 17	69°	"		"
	Apr. 2	72°	"		"
	Apr. 16	54°	"		"
	May 2	65°	"		"
	May 13	96°	"		"
	May 29	102°	"		"

HOPEWELL

LOCATION OF JOINTS



B.B. TRACK

			X		
	Jan. 20	33°	30.00		
	Feb. 3	66°	29.99		
	Feb. 19	15°	30.005		
	Mar. 4	80°	29.98		
S. Rail	Mar. 17	75°	29.985		
	Apr. 2	72°	29.98		
	Apr. 16	55°	29.99		
	Apr. 28	83° ‡	29.98		
	May 12	85° ‡	29.985		
	May 29	105°	29.98		

	Jan. 20	33°	
	Feb. 3	65°	
	Feb. 19	15°	
	Mar. 4	80°	
N. Rail	Mar. 17	75°	
	Apr. 2	72°	
	Apr. 16	55°	
	Apr. 28	83° ‡	
	May 12	85° ‡	
	May 29	105°	

W.B. TRACK

	Jan. 20	33°	
	Feb. 3	65°	
	Feb. 19	15°	
	Mar. 4	80°	
S. Rail	Mar. 17	75°	
	Apr. 2	72°	
	Apr. 16	55°	
	Apr. 28	83° ‡	
	May 12	85° ‡	
	May 29	105°	

	Jan. 20	33°	30.00		
	Feb. 3	65°	29.99		
	Feb. 19	15°	30.005		
	Mar. 4	80°	29.98		
N. Rail	Mar. 17	75°	29.985		
	Apr. 2	72°	29.98		
	Apr. 16	55°	29.99		
	Apr. 28	83° ‡	29.98		
	May 12	85° ‡	29.985		
	May 29	105°	29.96		

The actual movements of rails are twice as great as those shown on this plate.

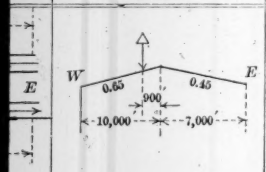
PLATE XXXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIII, No. 987.
WAGNER ON
CREEPING OF RAILS.

CREEPING RAILS. NEW YORK BRANCH. P. & R. RY. 1903.

Temperaure taken in sun.

LMONT

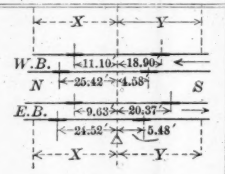
GRADE



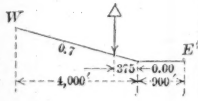
Y
30.00
"
"
"
"
"
29.995
"
30.00
29.99
"

NESHAMINY FALLS

LOCATION OF JOINTS



GRADE



W.B. TRACK

	Jan. 20	40°	X	Y
	Feb. 3	54°	"	"
	Feb. 19	18°	30.01	"
	Mar. 4	70°	29.99	"
N. Rail	Mar. 17	69°	"	"
	Apr. 2	72°	29.995	"
	Apr. 16	54°	29.99	30.01
	May 2	65°	"	"
	May 13	96°	29.995	29.995
	May 29	102°	29.99	30.00

	Jan. 20	40°
	Feb. 3	54°
	Feb. 17	18°
	Mar. 4	70°
S. Rail	Mar. 17	69°
	Apr. 2	72°
	Apr. 16	54°
	May 2	65°
	May 13	96°
	May 29	102°

E.B. TRACK

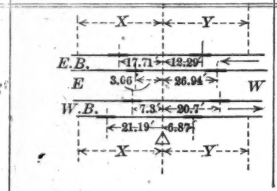
	Jan. 20	40°		
	Feb. 3	54°		
	Feb. 19	18°		
	Mar. 4	70°		
N. Rail	Mar. 17	69°		
	Apr. 2	72°		
	Apr. 16	54°		
	May 2	65°		
	May 13	96°		
	May 29	102		

	Jan. 20	40°	30.00	30.00
	Feb. 3	54°	"	"
	Feb. 19	18°	"	"
	Mar. 4	70°	"	"
S. Rail	Mar. 17	69°	"	"
	Apr. 2	72°	"	"
	Apr. 16	54°	"	"
	May 2	65°	"	"
	May 13	96°	"	"
	May 29	102°	"	"

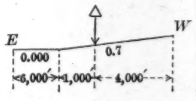
30.00
"
30.02
"
29.995
30.00
30.01
29.995
29.99
"

HOPEWELL

LOCATION OF JOINTS



GRADE



E.B. TRACK

	Jan. 20	33°	X	Y
	Feb. 3	65°	29.99	29.988
	Feb. 19	15°	30.005	30.00
	Mar. 4	80°	29.98	29.99
S. Rail	Mar. 17	75°	29.985	"
	Apr. 2	72°	29.98	"
	Apr. 16	55°	29.99	"
	Apr. 28	83°	29.98	29.98
	May 12	85°	29.985	29.975
	May 29	105°	29.98	29.97

	Jan. 20	33°
	Feb. 3	65°
	Feb. 19	15°
	Mar. 4	80°
N. Rail	Mar. 17	75°
	Apr. 2	72°
	Apr. 16	55°
	Apr. 28	83°
	May 12	85°
	May 29	105°

W.B. TRACK

	Jan. 20	33°
	Feb. 3	65°
	Feb. 19	15°
	Mar. 4	80°
S. Rail	Mar. 17	75°
	Apr. 2	72°
	Apr. 16	55°
	Apr. 28	83°
	May 12	85°
	May 29	105°

	Jan. 20	33°	30.00	30.00
	Feb. 3	65°	29.99	29.988
	Feb. 19	15°	30.005	30.02
	Mar. 4	80°	29.98	29.99
N. Rail	Mar. 17	75°	29.985	"
	Apr. 2	72°	29.98	30.005
	Apr. 16	55°	29.99	"
	Apr. 28	83°	29.98	29.99
	May 12	85°	29.985	"
	May 29	105°	29.96	29.98

REMARKS

BETHAYRES

⊗ Newtown crossing moved 1¼", all rails spiked & ballast in.

○ May 29, Bethayres. Ballast refilled in both tracks, but not between them.

HOFEWELL.

‡ Ballast all out of east track.

‡ " " " " both tracks.

The actual movements of rails are twice as great as those shown on this plate.

TABLE 1.—CREEPING RAILS. ATLANTIC CITY R. R. 1903.

To Accompany Plate XXX.

Location.	Alignment.	Bearing.	Rails.	Ballast.	Joints.	Cut or fill.	Traffic.	Remarks.
CLEMENTON: Between Station and Garden Lake. 20 ft. north of tel. pole 368.	Tangent.	N. 23° 45' W.	S. B. 90-lb. N. B. 90-lb.	Stone. Stone.	All joints spiked on ties.	Embankment 15 ft. high.	N. B. 27 Passenger, 4 Freight. S. B. 28 Passenger, 5 Freight. Daily.	Subgrade—gravel and sand. Alignment and surface good.
ALBION: Between station and 14 M. P. At tel. pole 458.	Tangent.	N. 37° 45' W.	S. B. 90-lb. N. B. 90-lb.	Stone. Stone.	All joints spiked on ties.	Embankment 12 ft. high.	N. B. 27 Passenger, 4 Freight. S. B. 28 Passenger, 5 Freight. Daily.	Subgrade—gravel and sand. Alignment and surface good.
FARMINGTON: 14 ft. north of north end of station platform.	Tangent.	N. 23° W.	N. B. 80-lb. S. B. 90-lb.	Cinders. Stone.	N. B., Spiked on ties. Standard splices. S. B., Spiked on ties. Bonzano splices.	Slight cut.	N. B. 17 Passenger, 2 Freight. S. B. 17 Passenger, 2 Freight. Daily.	Subgrade—gravel. Alignment and surface good. 45-ft. Rails on S. B. track.
PLEASANTVILLE: 36 ft. south of 51 M. P. on meadows.	Tangent.	N. 57° W.	N. B. 80-lb. S. B. 90-lb.	Cinders. Stone.	Same as above.	Embankment 8 ft. high.	Same as above.	Subgrade—gravel and rocks on salt marsh. Alignment and surface good. 45-ft. Rails on S. B. track.

TABLE 2.—Creeping Rails. New York Division, New York Branch, P. & R. Ry., 1903.

To accompany Plate XXXI.

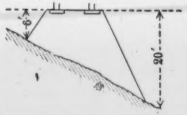
Location.	Alignment.	Bearing.	Rails.	Ballast.	Joints.	Cut or fill.	Traffic.	Remarks.
Bethayres: West end of bridge over Paul Brook.	Tangent.	N. 79° 30' E.	E. B., 90-lb. W. B., 90-lb.	Stone. Stone.	Spikes not in joints of E. B. track. W. B. track spiked at joints.	Embankment 15 ft. high.	E. B., 41 passenger, 19 freight. W. B., 40 passenger, 20 freight. Daily.	* W. B. track, ties respaced and spiked between Mar. 17 and Apr. 2 at Bethayres. † E. B. track, do. do. Apr. 16. Subgrade—earth fill on marsh. Alignment and surface good. Decided evidence of creeping on E. B. track. Will be spiked as soon as ties can be respaced.
Philmont: At 17 M. P.	Tangent.	N. 80° E.	E. B., 90-lb. W. B., 90-lb.	Stone. Stone.	All joints spiked on ties.	Cut 10 ft.	E. B., 41 passenger, 19 freight. W. B., 40 passenger, 20 freight. Daily.	Subgrade—stony soil. Alignment and surface good.
Neshaminy Falls: 200 ft. east of road crossing.	Tangent.	N. 78° E.	E. B., 90-lb. W. B., 90-lb.	Stone. Stone.	All joints spiked on ties.		E. B., 41 passenger, 19 freight. W. B., 40 passenger, 20 freight. Daily.	Subgrade—earth fill. Alignment and surface good. Previous creeping noticed in W. B. track.
Hopewell: 84 ft. west of west end of station platform.	Tangent.	N. 64° 30' E.	E. B., 90-lb. W. B., 90-lb.	Stone. Stone.	W. B. track joints spiked. Large number of unspiked joints in E. B. track.	Cut 10 ft.	E. B., 25 passenger, 18 freight. W. B., 24 passenger, 20 freight. Daily.	Subgrade—earth and soft. Alignment and surface good. No previous creeping noticed.

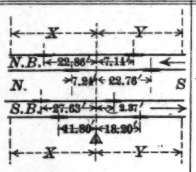
PLATE XXXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIII, No. 987.
WAGNER ON
CREEPING OF RAILS.

RECORD OF CREEPING RAILS. BETHLEHEM BRANCH P. & R. RY. 1903.

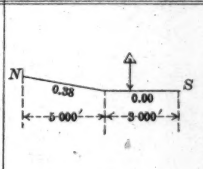
Temperature taken in sun.

QUAKERTOWN

LOCATION OF JOINTS



GRADE



N.B. TRACK

		X	Y
Jan. 19	16°	30.00′	30.00′
Feb. 2	45°	30.00′	30.00′
Feb. 18	15°	"	"
Mar. 3	53°	"	"
Mar. 17	49°	"	"
Apr. 1	70°	"	"
Apr. 17	68°	"	"
Apr. 28	75°	"	"
May 12	86°	30.00′	29.995′

E. Rail

W. Rail

Jan. 19	16°
Feb. 2	45°
Feb. 18	15°
Mar. 3	53°
Mar. 17	49°
Apr. 1	70°
Apr. 17	68°
Apr. 28	75°
May 12	86°

S.B. TRACK

Jan. 19	16°
Feb. 2	45°
Feb. 18	15°
Mar. 3	53°
Mar. 17	49°
Apr. 1	70°
Apr. 17	68°
Apr. 28	75°
May 12	86°

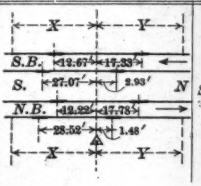
E. Rail

W. Rail

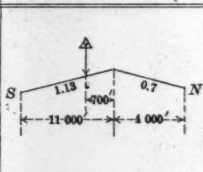
Jan. 19	16°	30.00′	30.00′
Feb. 2	45°	"	29.995′
Feb. 18	15°	"	"
Mar. 3	53°	29.995′	29.99′
Mar. 17	49°	29.99′	29.992′
Apr. 1	70°	29.975′	29.985′
Apr. 17	68°	29.98′	29.98′
Apr. 28	75°	29.96′	"
May 12	86°	"	"

NORTH WALES

LOCATION OF JOINTS



GRADE



S.B. TRACK

		X	Y
Jan. 19	27°	30.00′	30.00′
Feb. 2	47°	"	29.998′
Feb. 18	25°	"	30.01′
Mar. 3	57°	"	29.995′
Mar. 17	60°	"	30.00′
Apr. 1	76°	"	29.99′
Apr. 17	76°	"	"
Apr. 28	96°	29.99′	"
May 12	100°	30.00′	29.985′

W. Rail

E. Rail

Jan. 19	27°
Feb. 2	47°
Feb. 18	25°
Mar. 3	57°
Mar. 17	60°
Apr. 1	76°
Apr. 17	76°
Apr. 28	96°
May 12	100°

N.B. TRACK

Jan. 19	27°
Feb. 2	47°
Feb. 18	25°
Mar. 3	57°
Mar. 17	60°
Apr. 1	76°
Apr. 17	76°
Apr. 28	96°
May 12	100°

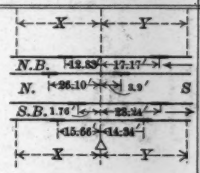
W. Rail

E. Rail

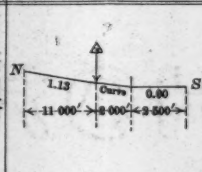
Jan. 19	27°	30.00′	30.00′
Feb. 2	47°	29.975′	30.015′
Feb. 18	25°	30.00′	30.005′
Mar. 3	57°	29.98′	30.00′
Mar. 17	60°	29.97′	"
Apr. 1	76°	29.97′	"
Apr. 17	76°	29.975′	"
Apr. 28	96°	29.97′	"
May 12	100°	29.97′	"

GWYNEDD.

LOCATION OF JOINTS



GRADE



N.B. TRACK

		X	Y
Jan. 19	40°	30.00′	30.00′
Feb. 2	51°	"	30.00′
Feb. 18	21°	30.013′	29.998′
Mar. 3	56°	29.995′	30.00′
Mar. 17	63°	30.00′	29.995′
Apr. 1	93°	29.99′	30.00′
Apr. 17	76°	"	29.995′
Apr. 28	101°	29.985′	30.00′
May 12	98°	"	29.995′

W. Rail

E. Rail

Jan. 19	40°
Feb. 2	51°
Feb. 18	21°
Mar. 3	56°
Mar. 17	63°
Apr. 1	93°
Apr. 17	76°
Apr. 28	101°
May 12	98°

S.B. TRACK

Jan. 19	40°
Feb. 2	51°
Feb. 18	21°
Mar. 3	56°
Mar. 17	63°
Apr. 1	93°
Apr. 17	76°
Apr. 28	101°
May 12	98°

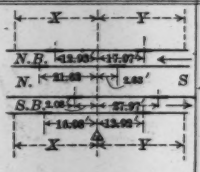
E. Rail

W. Rail

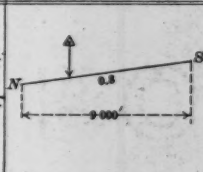
Jan. 19	40°	30.00′	30.00′
Feb. 2	51°	"	29.99′
Feb. 18	21°	29.995′	29.995′
Mar. 3	56°	"	29.98′
Mar. 17	63°	29.995′	"
Apr. 1	93°	"	29.97′
Apr. 17	76°	30.00′	"
Apr. 28	101°	"	"
May 12	98°	29.995′	"

ORELAND

LOCATION OF JOINTS



GRADE



N.B. TRACK

		X	Y
Jan. 19	35°	30.00′	30.00′
Feb. 2	52°	"	29.99′
Feb. 18	13°	30.003′	30.015′
Mar. 3	53°	30.00′	29.99′
Mar. 17	63°	"	29.985′
Apr. 1	76°	30.01′	"
Apr. 17	67°	30.005′	29.99′
Apr. 28	93°	"	29.985′
May 12	97°	30.00′	29.98′

W. Rail

E. Rail

Jan. 19	40°
Feb. 2	51°
Feb. 18	21°
Mar. 3	56°
Mar. 17	63°
Apr. 1	76°
Apr. 17	67°
Apr. 28	93°
May 12	97°

S.B. TRACK

Jan. 19	40°
Feb. 2	51°
Feb. 18	21°
Mar. 3	56°
Mar. 17	63°
Apr. 1	76°
Apr. 17	67°
Apr. 28	93°
May 12	97°

E. Rail

W. Rail

Jan. 19	40°	30.00′	30.00′
Feb. 2	51°	"	"
Feb. 18	21°	30.018′	30.01′
Mar. 3	56°	30.00′	29.992′
Mar. 17	63°	29.99′	29.995′
Apr. 1	76°	29.98′	"
Apr. 17	67°	29.99′	29.99′
Apr. 28	93°	29.98′	29.985′
May 12	97°	"	29.98′

The actual movements of rails are twice as great as those shown on this plate.

TABLE 3.—Creeping Rails. New York Division, Bethlehem Branch, P. & R. Ry. 1903.

To accompany Plate XXXII.

Location.	Alignment.	Bearing.	Rails.	Ballast.	Joints.	Cut or fill.	Traffic.	Remarks.
Quakertown: 30 ft. north of Hall signal, 825 ft. north of yard switches.	Tangent.	S. 25° E.	S. B., 90-lb. N. B., 80-lb.	Slag. Slag.	West rail of S. B. track not spiked on account of creeping. All other joints spiked to ties. Standard joints.	No cut or fill.	N. B., 12 passenger, 11 freight. S. B., 12 passenger, 11 freight. Daily.	Alignment and surface fair. Shortage of ballast between ties Subgrade—earth. Foreman reports west rail of S. B. track creeps to south. Last summer it moved 12 in. Ties at joints show creeping.
North Wales: 780 ft. south of 22 M. P. south of station.	0° 30′ curve. 4 223 ft. long.	S. 17° 30′ E. Tangent to curve.	S. B., 80-lb. N. B., 80-lb.	Slag. Slag.	All joints spiked on ties.	Rock cut 20 ft. deep.	N. B., 26 passenger, 12 freight. S. B., 26 passenger, 12 freight. Daily.	Alignment and surface fair. Shortage of ballast between ties. Subgrade—rock. No signs of creeping.
Gwynedd: 300 ft. south of stone arch bridge.	Tangent.	N. 26° 15′ W.	N. B., 80-lb. S. B., 80-lb.	Slag. Slag.	All joints spiked on ties.	Slight cut.	N. B., 26 passenger, 11 freight. S. B., 26 passenger, 12 freight. Daily.	Alignment and surface fair. Shortage of ballast between ties. No signs of creeping.
Oreland: 375 ft. north of road crossing north of station.	Tangent.	N. 56° 30′ W.	N. B., 80-lb. S. B., 80-lb.	Slag. Slag.	Some joints spiked to ties, others are not.	Cut 5 ft. deep.	N. B., 26 passenger, 12 freight. S. B., 26 passenger, 14 freight. Daily.	Alignment good. Surface fair. Shortage of ballast between ties. East rail, N. B. track, creeps north, and is shown by ties. Rails on N. B. track close together at station and open out as you go north.

TABLE 4.—CREEPING RAILS. READING DIVISION, FRACKVILLE BRANCH, P. & R. RY. 1903.

To accompany Plate XXXIII.

Location.	Alignment.	Bearing.	Rails.	Ballast.	Joints.	Cut or fill.	Traffic.	Remarks.
HEAD OF GRADE: At telegraph pole 285.	Tangent.	N. 30° W.	S. B., 90-lb. N. B., 90-lb.	Cinders. Cinders.	Both rails of N. B. track spiked. Occasional joints on S. B. track are spiked, say, one-third (?). Standard splices.	Cut on side of mountain.	N. B., 7 passenger, 65 freight. S. B., 7 passenger, 65 freight.	Alignment and surface fair for conditions. Foreman reports difficulty with creeping on S. B. track. He cut five places last year. Rails last but 18 months. Evidence of movement at joint ties.
BROAD MOUNTAIN: 360 ft. north of the station.	1° 30′ curve.	N. 5° W. Tangent to Curve.	S. B., 90-lb. N. B., 90-lb.	Cinders. Cinders.	Both rails of N. B. track spiked. On S. B. track a number not spiked. Standard splices.	Cut on side of mountain.	Same as above.	See above note.
SHAMOKIN DIVISION, MAHANOY AND SHAMOKIN R. R., P. & R. RY. 1903.								
LOCUST DALE JUNCTION: 68 ft. north of switch in S. B. track.	Tangent.	S. 80° W.	S. B., 90-lb. N. B., 80-lb.	Cinders. Cinders.	All joints spiked on ties.	No cut or fill.	S. B., 11 passenger, 14 freight, 41 light engines. N. B., 11 passenger, 30 freight. Daily.	Surface and alignment good. Decided evidence of creeping on S. B. track, where foreman reports trouble, especially in summer.
LOCUST SUMMIT: South of bridge, below the station.	Tangent.	N. 88° E.	S. B., 90-lb. N. B., 80-lb.	Cinders. Cinders.	All joints spiked on ties.	Cut 15 ft. deep.	Same as above.	Same note as above.

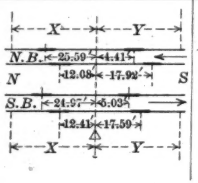
PLATE XXXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIII, No. 987.
WAGNER ON
CREEPING OF RAILS.

CREEPING RAILS. READING DIVISION. FRACKVILLE BRANCH.- P. & R. R. R.

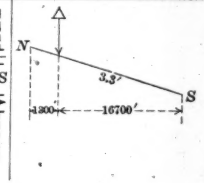
SHAMOKIN DIVISION. MAHANOY AND SHAMOKIN R. R.- P. & R. R. R.

HEAD OF GRADE. FRACKVILLE BR.

LOCATION OF JOINTS.



GRADE.



N.B. TRACK

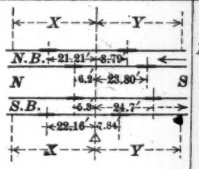
			X	Y
E. Rail	Feb. 16	26°	30.00	30.00
	Mar. 2	44°	29.98	29.99
	Mar. 16	39°	30.00	30.00
	Mar. 31	58°	29.995	"
	Apr. 13	49°	30.00	"
	Apr. 27	80°	29.975	29.99
	May 11	84°	29.97	"
	Jun. 2	88°	"	29.985
	Sep. 28	61°	30.00	30.00
W. Rail	Feb. 16	26°		
	Mar. 2	44°		
	Mar. 16	39°		
	Mar. 31	58°		
	Apr. 13	49°		
	Apr. 27	80°		
	May 11	84°		
	Jun. 2	88°		
	Sep. 28	61°		

S.B. TRACK

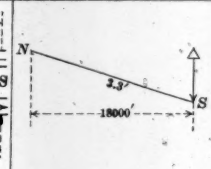
E. Rail	Feb. 16	26°		
	Mar. 2	44°		
	Mar. 16	39°		
	Mar. 31	58°		
	Apr. 13	49°		
	Apr. 27	80°		
	May 11	84°		
	Jun. 2	88°		
	Sep. 28	61°		
W. Rail	Feb. 16	26°	30.00	30.00
	Mar. 2	44°	Worn out	Worn out
	Mar. 16	39°	30.00	30.00
	Mar. 31	58°	"	"
	Apr. 13	49°	"	29.99
	Apr. 27	80°	"	30.01
	May 11	84°	"	"
	Jun. 2	88°	29.99	"
	Sep. 28	61°	30.00	30.00

BROAD MOUNTAIN. FRACKVILLE BR.

LOCATION OF JOINTS.



GRADE.



N.B. TRACK

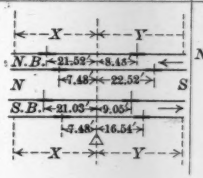
			X	Y
E. Rail	Feb. 16	25°	30.00	30.00
	Mar. 2	42°	29.985	29.99
	Mar. 16	39°	29.995	"
	Mar. 31	64°	30.00	30.00
	Apr. 13	52°	29.99	"
	Apr. 27	75°	"	29.99
	May 11	77°	"	29.98
	Jun. 2	78°	29.98	29.99
	Sep. 28	64°	30.00	30.00
W. Rail	Feb. 16	25°		
	Mar. 2	42°		
	Mar. 16	39°		
	Mar. 31	64°		
	Apr. 13	52°		
	Apr. 27	75°		
	May 11	77°		
	Jun. 2	78°		
	Sep. 28	64°		

S.B. TRACK

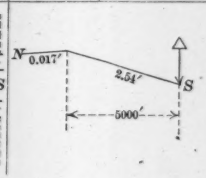
E. Rail	Feb. 16	25°		
	Mar. 2	42°		
	Mar. 16	39°		
	Mar. 31	64°		
	Apr. 13	52°		
	Apr. 27	75°		
	May 11	77°		
	Jun. 2	78°		
	Sep. 28	64°		
W. Rail	Feb. 16	25°	30.00	30.00
	Mar. 2	42°	Worn out	"
	Mar. 16	39°	30.02	"
	Mar. 31	64°	30.025	29.995
	Apr. 13	52°	30.02	"
	Apr. 27	75°	30.01	30.00
	May 11	77°	30.00	"
	Jun. 2	78°	"	"
	Sep. 28	64°	"	"

LOCUST DALE JUNC. SHAMOKIN DIV.

LOCATION OF JOINTS.



GRADE.



N.B. TRACK

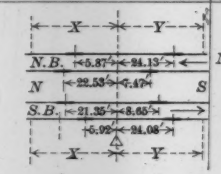
			X	Y
E. Rail	Feb. 16	30°	30.00	30.00
	Mar. 2	51°	29.99	29.99
	Mar. 16	45°	"	"
	Mar. 30	35°	—	—
	Apr. 13	50°	29.99	29.99
	Apr. 27	80°	29.98	"
	May 11	94°	"	"
	Jun. 2	101°	29.975	"
	Sep. 28	57°	—	—
W. Rail	Feb. 16	30°		
	Mar. 2	51°		
	Mar. 16	45°		
	Mar. 30	35°		
	Apr. 13	50°		
	Apr. 27	80°		
	May 11	94°		
	Jun. 2	101°		
	Sep. 28	57°		

S.B. TRACK

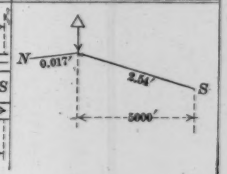
E. Rail	Feb. 16	30°		
	Mar. 2	51°		
	Mar. 16	45°		
	Mar. 30	35°		
	Apr. 13	50°		
	Apr. 27	80°		
	May 11	94°		
	Jun. 2	101°		
	Sep. 28	57°		
W. Rail	Feb. 16	30°	30.00	30.00
	Mar. 2	51°	29.99	"
	Mar. 16	45°	30.00	"
	Mar. 30	35°	—	—
	Apr. 13	50°	29.99	30.00
	Apr. 27	80°	"	"
	May 11	94°	29.98	29.99
	Jun. 2	101°	"	"
	Sep. 28	57°	29.985	30.00

LOCUST SUMMIT. SHAMOKIN DIV.

LOCATION OF JOINTS.



GRADE.



N.B. TRACK

			X	Y
E. Rail	Feb. 16	35°	30.00	30.00
	Mar. 2	61°	"	29.985
	Mar. 16	43°	"	30.00
	Mar. 30	35°	—	—
	Apr. 13	52°	—	30.00
	Apr. 27	95°	—	"
	May 11	96°	30.00	"
	Jun. 2	93°	"	"
	Sep. 28		Stake lost	
W. Rail	Feb. 16	35°		
	Mar. 2	61°		
	Mar. 16	43°		
	Mar. 30	35°		
	Apr. 13	52°		
	Apr. 27	95°		
	May 11	96°		
	Jun. 2	93°		
	Sep.		Stake lost	

S.B. TRACK

E. Rail	Feb. 16	35°		
	Mar. 2	61°		
	Mar. 16	43°		
	Mar. 30	35°		
	Apr. 13	52°		
	Apr. 27	95°		
	May 11	96°		
	Jun. 2	93°		
	Sep.		Stake lost	
W. Rail	Feb. 16	35°	30.00	30.00
	Mar. 2	61°	"	29.985
	Mar. 16	43°	"	30.00
	Mar. 30	35°	—	—
	Apr. 13	52°	—	30.00
	Apr. 27	95°	—	"
	May 11	96°	30.00	"
	Jun. 2	93°	"	"
	Sep.		Stake lost	

The actual movements of rails are twice as great as those shown on this plate.

DISCUSSION.

Mr. Lindenthal.

GUSTAV LINDENTHAL, M. AM. SOC. C. E. (by letter).—This subject has been of much interest to the writer for many years, particularly in connection with track over bridges, and has led him to an investigation of the creeping of rails, generally, and the formulation of a theory, which he has published,* and from which he desires to quote, as follows:

"As is well known, rails creep in the direction of the traffic, whether up grade or down grade. On single track, rails creep in the direction of heavier traffic; otherwise they creep down grade.

"The force with which a track is creeping seems almost irresistible. Notching the rail base for the spikes was one of the means employed to stop it. But the spikes were gradually sawed into, pulled out or sheared off by the creeping rails. Later, when the practice of notching was given up, because steel rails were apt to break at the notched places, spiking of the angle bars at the rail joints was resorted to. Special angle brackets or straps bolted to the rail between joints are also used. But all in vain.

"Several explanations have been given for the phenomenon, but none accounts for all its peculiarities satisfactorily. My own view is that creeping is caused by the push of the rolling friction of the wheels on the rails. If, for illustration, we imagine the cars mounted on skids instead of wheels, the creeping would be greater in the proportion as sliding friction is greater than rolling friction. The drivers of the locomotives are the only wheels exerting a pull on the rails, which for the time being are pinned down and so cannot move. The pull is transmitted by friction through the ties to the ballast and earth underneath, the whole forming an anchorage—but a traveling anchorage—always located under the locomotive, whether pulling by adhesion, or through cog-wheels on a rack rail.

"All other wheels behind the locomotive exert a push on the rails through rolling friction in the direction of the train.

"The coefficient of rolling friction varies with the velocity of the train from nearly one per cent. for slow velocities to four per mill for trains at and over 30 miles per hour. It also varies on curves. The average value for fast trains on tangents and curves may be assumed for illustration at 5 per mill. A car wheel loaded with 10 000 lbs. will thus exert a push of 50 lbs. on the rail.

"As the friction between the loaded rail and the ties—disregarding the holding power of the any way imperfectly fitting and usually loose spikes—is at least 30 times greater than the rolling friction of the wheels, the rail could not possibly move or creep under a smoothly rolling load.

"But the wheels do not roll smoothly and moreover they do not roll over continuous rails. The rails are in lengths from 30 to 60 ft., with space between the ends for expansion. The wheels get upon each rail length suddenly, and instead of a quiet push or a static force of 50 lbs. per wheel, there is a horizontal blow or a dynamic force, the effect of which increases as the square of the velocity of the train.

"It should be understood that this horizontal blow is not caused by the depression of the rail ends or by the space left for expansion

* *Railroad Gazette*, September 29th, 1899.

Mr. Lindenthal.

between them, although both these features, when they exist, undoubtedly augment its force. It is unavoidable with the smoothest and any kind of rail joint (including the joint in which the outside splice bar carries the wheels over the rail ends) because the longitudinal strain is set up suddenly in each rail length, and cannot be transmitted to the next rail, unless the rail ends were tightly pressed together, equivalent to a continuous rail.

"Taking 40 miles per hour, a usual speed for fast freight trains, equal to 60 ft. per second, the impact from the rolling friction of the wheel will be $50 \times 60^2 = 180\,000$ foot-pounds, which in effect is the same as if an iron ram, weighing one ton, would hit the end of the rail with a velocity of $9\frac{1}{2}$ ft. per second. The car wheels being spaced 5 to 20 ft. apart, will deliver upon the receiving end of a 30-ft. rail from 3 to 6 such blows per second. If the rail is not rigid and smooth, if, on the contrary, by reason of defective or yielding supports, it deflects and by its resilience causes the wheels to rebound and ricochet, so that they no longer roll with uniform pressure, then each wheel will hit the rail as many additional horizontal blows as it makes jumps or rebounds. A rebound every few feet is an ordinary occurrence on spiked track. A 30-ft. rail may thus receive from 30 to 60 glancing blows per second, and from a train of 40 cars a total of 1 500 to 3 000 blows, varying in effect from a few foot-pounds to 180 000 and more per blow, lengthwise with the rail and in the direction of the train.

"With this explanation it is not difficult to understand why the creeping should seem irresistible, the rails shearing off spikes and bolts and wandering up steep grades. In principle, to make it plain, it is the same dynamic effect with which the blows from a 10-lb. hammer will suffice to overcome the friction (of over 5 000 lbs.) of a rail spike in an oak tie.

"A rail may be held down by spikes and by the moving loaded cars, but it will nevertheless be displaced by their dynamic effect in the manner stated.

"Each rail moves individually and independent of the adjacent rails. The play left for expansion at the joints facilitates the creeping. There could be no creeping with a continuous rail.

"All observed facts can now be satisfactorily explained. There is no creeping in switch yards or on side tracks, on which trains move slowly and in opposite directions. Compared on the basis of ton-mileage, the rate of creeping is greater on tracks for fast trains, and with short or light flexible rails, and on poorly ballasted and yielding foundations. Every cause which induces or increases rebounding of the wheels (as poor rail joints) will increase creeping.

"On down-grades, where the locomotive is not pulling, the effect of its wheels will be added to that of the car wheels, and on that account and by reason of the greater velocities on down grades, the creeping will be greater. So also the outside rails on curves creep faster than the inside rails, because exposed to greater pressure and friction, causing harder glancing blows."

Those who are interested in the subject may follow the writer's article further, on the remedy for creeping, etc.

The only correction that the writer would make in the foregoing citation is in the coefficient of rolling friction. Five per mill is probably true enough for rough defective track; on first-class track, with 100-lb. rails, the rolling friction between wheel and rail may be only

one-tenth as large. Data on rolling friction are still unreliable and divergent; but with poor rail joints, leaving the receiving rail and sticking up like a step, it will receive something more than a glancing blow. The assumption of 5 per mill rolling friction, in such case, will not require reduction. Mr. Lindenthal.

The explanation of creeping, cited by Mr. Wagner from Camp's "Notes on Track," that the principal cause is "the wave motion in the rails set up by moving trains," is, in view of the foregoing, as little satisfactory as would be the explanation for the progress of the locomotive being caused by crank motion. It stops too far away from the primary cause, which is of a dynamic nature. If the rails consisted of discontinuous high stiff girders on solid continuous bearings, as, for instance, on longitudinal concrete beds, which construction could not be subject to wave motion, there would still be creeping of rails, although at a diminished rate. As long as rails may move on their bearings from temperature changes, so long they may move from the rolling friction of the wheels, which is the explanation for creeping, as stated above.

W. M. CAMP, M. AM. SOC. C. E. (by letter).—This paper takes up an old, though timely, subject. The misadjustments in track caused by creeping rails cannot be considered new experience. They have been a source of trouble to trackmen since the early days of railroading, to a greater or less extent; and, while the causes, perhaps, may be more widely understood at the present time than was the case a generation ago, there still remain some things to learn concerning the phenomena of rail movements under traffic, in order to seek intelligently the most effective means or methods of prevention. As the creeping action is always more pronounced on double, than on single, track, other conditions being the same, the subject gains importance with time, owing to increase in the mileage of second track; and increase in the weight of rolling stock is another cause which aggravates the trouble. Mr. Camp.

Although, in years past, there has been considerable discussion of rail creeping and the conditions responsible for it, yet the subject was seldom or never investigated with that thoroughness which is characteristic of engineers when setting out to overcome difficulties. Therefore, it is a matter of no little satisfaction that Mr. Wagner has collected exact measurements and other data of rail creeping, in connection with careful observations of the attending conditions. It is only through the publication of such results, and discussion of the whole question from varied lessons of experience, that the problem can be thoroughly comprehended.

The records found in this paper, while not exhibiting extreme cases of rail movement, nevertheless present a variety of conditions and afford good opportunity for studying the subject. The greatest

Mr. Camp. movement seems to have been at Bethayres, on the New York Branch, and at "Head of Grade," on the Frackville Branch. At the former point the movement was 3¼ in. in the south rail of the east-bound track, in the direction of the traffic, in a period of 4 months and 9 days; in the north rail of the same track the movement was 2⅝ in. during the same time.

The difference in movement between the two rails of the same track is only a matter of common occurrence. The larger movement of the south rail may have been caused by its location nearer the outside of the roadbed, where there would be more spring in the bank, and consequently a greater amplitude of wave motion. In some cases, also, the spikes on one rail of a track may happen to be staggered in the manner which secures the most effective cross-binding action on the rail, with a tendency to hold it against creeping. This occurs where the outside spike leads in the direction of the creeping.

The roadbed at this point is an earth fill over a marsh, a combination of two conditions quite conducive to rail creeping. The rails in the two tracks were of the same weight, and the traffic in the two directions was practically the same, in regard to the number of both passenger and freight trains, but the rails in the west-bound track were held by slot-spiking at the splice-bars, whereas such was not the case in the east-bound track. This fact will account, in some degree, for the smaller movement of the rails in the west-bound track, where the creeping amounted to only from ⅝ to ⅞ in. during the period of the observations.

Another cause for the difference might have been a variation of speed in the trains in the two directions. The point of observation was in a sag, at the foot of a grade of 0.7 %, about 3 miles long, meeting a level stretch of track, 3000 ft. long, across the marsh. It is reasonable to suppose that at least the freight trains on the east-bound track, approaching the point of observation on a descending grade, would pass the point at higher speed than trains of the same class on the west-bound track. The comparison of the creeping action on the two tracks, therefore, is corroborative of explanations which are well understood.

The case next in importance, measured by the extent of the creeping action, seems to have been at "Head of Grade," on the Frackville Branch, where the west rail of the south-bound track crept 3⅜ in. during the time between February 16th and September 28th, but, during the same time, the east rail of the same track crept only 1¼ in. The creeping on the north-bound track at the same point was unimportant. The grade at this point is 3.3 % for a distance of 3.16 miles. Examination of Table 4 shows that the joints on both rails of the north-bound track were slot-spiked, whereas on the south-bound track only occasional joints were held in this way. The traffic, both pas-

senger and freight—but with the freight trains greatly predominating—seems to have been the same on both tracks. It might be expected, however, that the creeping force would be much greater on the south-bound track, on which the trains run with the grade, and on which faster speeds are undoubtedly made than on the north-bound track, where the train movements are against the grade. It is also stated that the freight trains on the north-bound track consisted almost entirely of empty cars. Here, again, therefore, the observations and accepted theories applying to the conditions are in harmony. Mr. Camp.

At Broad Mountain the conditions respecting grades and traffic seem to be substantially the same as at "Head of Grade," with similar, but not as serious, results.

It appears from the diagrams that in several places the movement of the rails was opposite to the direction of the traffic. In every case of this kind, however, the extent of the movement was small, being most at Hopewell, where it was $\frac{3}{4}$ in. in the north rail of the east-bound track and $\frac{1}{2}$ in. in the north rail of the west-bound track. At North Wales both rails of the north-bound track, apparently, moved in opposition to the traffic, the maximum contrary movement there being about $\frac{9}{16}$ in.

At a number of other points there was apparently a slight movement in the direction opposite to that of the traffic, but in every case the extent of the movement was so small, and the movement of the rails in either direction, at those points, so small, that it may reasonably be doubted whether rail creeping, proper, took place at all. The writer is inclined to think that the movement in such cases was only the effect of local expansion or contraction, from change of temperature, which might have been stronger than the tendency of the rails to creep under the traffic.

Rail creeping causes trouble and expense in various ways. The movement shoves joint ties off their tamped beds upon loose ballast, which affords inferior support; the slewed ties must be respaced, to put them square with the track or to restore the proper space intervals; frogs are sometimes crowded out of alignment; and the creeping action closes joint openings necessary for the free expansion of the rails with rise of temperature. Other well-known evils might be mentioned, but the foregoing will suffice to indicate the relative importance of this matter in track work.

In seeking to arrest rail creeping, it is expedient, for practical considerations, to be able to connect the effect with the cause, for it is well understood that the tendency for rails to creep is greater in some localities than in others. Observers have come to know that the character of the ground formation or roadbed is one of the most important conditions accountable for the creeping movement. In using means of prevention, therefore, it is essential that knowledge of conditions

Mr. Camp. of this kind, in some detail, should enter into the considerations. In practice, trackmen have found it possible to stop the creeping at some points, while at other points the same methods were entirely inadequate to overcome the trouble or even alleviate the situation in any material respect. Hence the economical aspects of the question also call for knowledge of the local conditions. It follows from this that a concentration of appliances at those points where the tendency to creeping is strongest may be more effectual than an application of any particular device uniformly at all points where creeping occurs. That is to say, the creeping of rails at all points along a stretch of track of assumed length might be due to conditions which exist along only a portion of the distance. The creeping action at the particular points or localities where the propelling force is engendered may be strong enough, if not opposed, to shove the line of rails over the whole distance considered. The records and diagrams presented by Mr. Wagner show that the rail creeping at some points was negligible, while at other points it was not entirely preventable by the means applied to overcome it, although they might have retarded it to some extent.

In his paper, Mr. Wagner has been kind enough to refer to certain characteristic facts of rail creeping put into formulated expression by the writer after some years of reflection on experience. It will not be necessary, therefore, to repeat the writer's opinions on the general principles of the question except where special comment is intended. One of the principles laid down is that rails creep more on newly-made embankments than on old roadbed which has become solidly compacted. By way of illustration, a case will be cited which came within the writer's experience at one time on the Pennsylvania and New York Division of the Lehigh Valley Railroad. The original roadbed on this line was the tow-path of the abandoned Pennsylvania and New York Canal, which had become well settled under the tramping of boating teams, and from weather exposure for many years. Eventually, the road was double-tracked, the second track at almost all points being laid upon a new embankment made by filling upon the bed of the abandoned canal. At one location, in particular, covering a stretch of track some 4 miles in length, this new embankment was continuous, and upon it was laid the new north-bound track, the old track then being used for the south-bound traffic. The grades were light enough to be fairly negligible in the present connection, and the number of train movements was about the same in each direction, the weight of traffic being somewhat heavier north bound, owing to the movement of loaded coal trains in that direction, although the speed of these loaded north-bound trains was slower than the empty south-bound coal trains The height of the new embankment was about 8 ft. and upward. On both tracks the joint splices were slot-spiked to the

ties, and both tracks were ballasted with gravel of good quality. The exact weight of the rails on the two tracks cannot now be recalled, but those on the north-bound track were new and of heavier section than the old rails of the south-bound track, the weight of which was somewhere between 50 and 60 lb. per yd. Mr. Camp.

As soon as the road began to be operated as double track, excessive creeping was observed on the north-bound track, and all measures taken to stop it were ineffective. In six months, during the summer, the rail movement, by actual measurement, was 22 in. The creeping action was so strong that both ties at all the (suspended) joints were carried bodily with the rails, so that respacing of these ties became necessary long before the whole movement noted had taken place. On the old roadbed (south-bound track) the creeping movement during the same time was only 3½ in. During the following year the creeping on the north-bound track was not nearly as much, and the comparison of the rail movements of the two tracks was not nearly as marked, which was undoubtedly due to the fact that, as the new roadbed became consolidated, the force causing the creeping gradually decreased.

In regard to grades, the writer is of the opinion that they are a factor in rail creeping only as far as they influence the speed of trains. On single track he has observed rail creeping both up and down grade, and, in one case, which is worth citing, the rails crept in both directions on the same grade. This was on the Seattle, Lake Shore and Eastern Railroad (now the Seattle and International Railway, a branch of the Northern Pacific Railway), on a grade about 5 miles long. Over nearly the whole length of this grade, from the top, the rails crept down hill, presumably because of the faster speeds in that direction. For a distance of about ½ mile from the foot of the grade, however, the rails crept up hill. The explanation was found in the high speed of loaded logging trains "running for the hill" when beginning to ascend the grade. At the foot of the grade the creeping movement up hill amounted to 2 in., in one season; but, farther up the grade, the creeping action gradually grew less and less, until, as already stated, it died out altogether in a distance of about ½ mile. In this case it seemed clear that, as the speed of the heavily loaded trains slackened in working against the grade, the rail creeping up grade gradually decreased in correspondence, until a point was reached where it was counteracted by the opposite creeping action caused by the faster speeds on the down grade.

F. S. Stevens, M. Am. Soc. C. E. (by letter).—As the writer understands the matter, the term, "creeping," only applies to those cases where the rails are moved by external forces; and movement which is due to expansion and contraction should not be considered. The only force, then, that can produce the trouble is the moving train, and, as Mr. Stevens.

Mr. Stevens. the friction or adhesion will hold the rails in place on the ties, it follows that this movement must occur where the rail is not loaded and is practically free from contact with the ties, and, therefore, it is due to the wave motion caused by the weight of the train passing over a roadbed which yields more or less to the compression and is somewhat elastic. The movement, then, must precede the train, and will increase directly with the elasticity of the roadbed and the condition of the track as to surface. The writer has had charge of track over ground which, apparently at some time, had been swamp, and the soft alluvial formation, of great depth, carried the embankment of gravel, not exceeding 6 to 8 ft. high; but this comparatively thin layer of gravel was subjected to great vibration under passing trains, and the wave in front of the engine was very perceptible. The result was that frequently the splice-plates were broken at the center, and the rails separated from 6 to 12 in., the splice-plates evidently having cracked some time before, and the joint parted in this way; and if there were no cracked plates to part and thus relieve the strain, the bolts were sheared off and the same opening appeared. The strange fact, in connection with the parting of these splices, was that the joints in the track on the hard ground adjoining did not close, but remained open for a distance of ¼ mile or more from the old swamp, and there the rails ran close together and showed a tendency to buckle. The movement was in the direction of the heavier traffic and up grade.

Similar conditions have been found where new ties have been "spotted" in at intervals and have not received proper attention within reasonable time afterward. These, becoming loose, hung by the spikes and gave the rails so little support that violent wave motion was set up and the consequent creeping took place.

The effect of loose ties is more noticeable where the tracks are curved, for where the traffic is heavy and slow, and trains are long, and where the track is surfaced for high speed, the load shifts to the low rail and the wheels that travel on the high rail must slip; and, in addition to the wave motion, there is the dragging effect of the sliding wheels, with the result that the outer rail of the curves creeps and the inner rail does not, to any great extent.

Loose spice-plates are also a factor in the creeping of rails, and, where other conditions are good, loose splices alone will cause some movement. The remedy is to prevent, as far as possible, all wave movement by using rails of sufficient weight to carry the loads without appreciable deflection, build firm and unyielding roadbeds, and use enough good ballast to distribute the load properly and see that it is kept open, so that all water may escape quickly and without saturating and softening the roadbed; then keep all ties tamped to a true and uniform bearing, each tie carrying its proper proportion of the

load. The rails in such track will creep very little, and no trouble will be found in keeping the proper expansion allowance at the joints, if the splices are kept uniformly tight. Mr. Stevens.

The writer does not believe that with tracks there has ever yet been any trouble which could be traced to excessive length of rail, but there has been a great deal of trouble caused by creeping rails which crept because they were neglected, or the ties were loose, the joints were loose, the surface poor, or the men in charge incompetent. When engines and cars ride smoothly, without jolt, jar, sway, or lurch, the rails are not creeping to any great extent. It is poor track which does the creeping, unless the trouble lies below the bottom of the roadbed. Drainage is the "whole thing" in making and maintaining good track which will hold its surface and stay where it is placed, and the cost of maintenance varies, as to rail and labor, very nearly in proportion to the tonnage moved. Eliminate the wave motion by thorough drainage, sufficient weight of rail, good ties, and good surface, with tight joints, and no fear need be entertained on account of trouble caused by creeping rails.

HUNTER MCDONALD, M. AM. SOC. C. E. (by letter).—The writer's experience with creeping rails has been confined almost entirely to single track, while the observations described by the author were made exclusively on double track. Mr. McDonald.

The creeping of rails on single track presents also many apparently inexplicable phenomena. It has been the writer's experience that, no matter what the direction of the preponderance of traffic, the rail will often creep down grade in both directions until all the joints in the sag are closed up. Gravity, no doubt, has much to do with this, but the writer believes that the temperature has also a large share in it. The rail is first laid with proper provision for expansion. As expansion takes place the rail moves more easily down the grade. When contraction follows, the rail is again drawn down the grade rather than up.

The creeping of rails against the current of traffic, as shown in many instances on the author's cards, often occurs for a short period, but it seldom continues unless some condition of loose track, easily discernible, brings it about. This creeping is probably due to temperature changes.

The writer recalls many instances of creeping rails, one of which may be worth mentioning. It was on a trestle about 4 000 ft. long. Near its middle the track was a level tangent. The south half was on a 1.5% grade, ascending from near the middle, and included a long 6° curve to the east. The north half was on a 0.6% grade, ascending from near the middle, and included two short, sharp curves near the north end, reversing on a point. The excess of traffic was southward. A Wharton switch turned out near the middle on the west side, the

Mr. McDonald. frog being rigidly fastened to the stringers. In earlier days the trestle had few braces in the direction of the track, and vibrated greatly under trains. The east rail crept northward for years at an average rate of $\frac{3}{4}$ in. per day. Efforts to stop it by fastening the rail always resulted in shearing the bolts or moving the ties, no matter how well fastened down. Until the method of using pointed rails was adopted, it was the practice to put in long rails at the south end and short ones at the north end every Saturday afternoon. The west rail never gave any trouble whatever from creeping in either direction. It was on the outside of the long 6° curve. All creeping was effectually stopped when heavier stringers were put in and every eighth bent braced securely by two heavy pieces of timber fastened on each side of the cap and running to the sills of the second bents on each side.

It has been the writer's experience that the rail always creeps on, and in the vicinity of, long trestles, especially those providing for large waterway across bottoms, and in which, on account of drift-wood, it is not practicable to maintain much longitudinal bracing.

When some method of fastening the rail securely to the tie is devised, more rigid track can be maintained, and trouble from creeping greatly reduced.

The anti-creeping devices, designed upon the wedge principle, will hardly take care of cases where the rail creeps in both directions, and will not be needed, to any great extent, when the present method of spiking gives way to one that will hold the rail tight down to the tie.

Dr. Dudley. P. H. DUDLEY, ESQ. (by letter).—The measurements of the creeping of rails, by Mr. Wagner, are important, because they cover so many locations and conditions of service. These will lead to others of as high a degree of excellence, and the principal conditions incident to creeping rails, some of which are local, will become evident.

When the writer's "questions" for the International Railway Congress were prepared and sent to the railroads, it was expected that a number of similar observations would be secured from several railroad companies. The replies, without measurements, were so discordant, owing to different conditions, that they could not be used to determine the local causes as to the creeping of the rails. After the questions had been sent out, the Permanent Committee of the International Railway Congress restricted the reports to a less number of pages than expected; therefore, detailed observations could not be reported, on subsidiary subjects. It is well that these are presented where they will receive adequate publication and discussion.

In Mr. Wagner's records of the Atlantic City Railroad, the creeping of the rails was not decided, for trains exceeding one mile per minute, from January 14th to May 7th, at Clementon, Albion, Farmington, and in part at Pleasantville, the movements being due principally to

the elongation and contraction from thermal stresses. From May 7th to November 24th, the movements were larger. The details, as to gradients and character of the track, are given for all the different points of observation. At Clementon, on the south track, to May 7th, the west or right rail had moved in the direction of the traffic 0.26 in. From May 27th to November 24th, a part of this was lost, the rail moving in the opposite direction, the final movement being 0.20 in., compared with the observation of January 14th. The east or left rail did not show much movement until after May 17th, when the total movement to the left was 0.28 in. Dr. Dudley.

On the north-bound track, to May 7th, the movement of the left rail seems to have been due principally to the adjustment of thermal stresses, and by May 27th it had moved against the traffic 0.16 in., but by November 24th it had moved 0.86 in. with the traffic.

The right or east rail, on May 27th, had moved 0.08 in. against the traffic, though by November 24th it had moved 0.20 in. with the traffic. This, under high-speed trains, is a limited amount.

At Albion the movement of the rails on the south-bound track was down grade against the traffic. On the north-bound track it was down grade also, though the total movement was slight. From May 7th to November 24th, for the left rail it was 0.36 in. and for the right rail 0.42 in.

At Farmington (level track) the right rail on November 10th had moved 0.1 in. with the traffic, the left rail 0.6 in. On the south-bound track, both rails, from May 8th to November 10th, moved only 0.22 in.

Examination of the figures giving the movement of the joints on either side of the observed lines shows, by the rails rendering in the splice-bars, that more or less motion was taking place. The friction of the 90-lb. splice-bars against rendering, in the testing machine at Watertown, was about 4 500 lb. per lin. in., or 63 000 lb. for 14 in. in one rail end, when the bolts were tightened up as by the trackmen.

At Pleasantville, on the north-bound track, from January 15th to November 10th, the right rail moved 0.36 in. with the traffic, and the left rail 1.14 in. On the south-bound track the left rail moved 1.80 in. with the traffic, the right rail moved only 0.56 in. At these two locations the rails are 45 ft. in length.

It is usually found that the 45 and 60-ft. rails are slightly looser under the spikes than the shorter rails.

The excess tendency of the right or left rail to move under the fast trains is indicated there by the greater movement for the left rail. This applies only to similar conditions, for the observations at other points show the greater creeping by the right rail.

The movement of the cross-ties is not noted, but was slight, if any, except for those to which the angle plates at the joints were spiked,

Dr. Dudley. the rails running under the spikes. Where the ballast is out, the cross-ties move, or permit more rapid running of the rails.

In the winter the tendency of the thermal stress is to cause tension in the metal of the rails, while in the summer it is compression, before or while the rail ends are rendering in the splice-bars, for adjustment. In the first condition the rails are not as loose as a continuous member under the passing locomotives and cars, and the unit fiber strains as a rule are not found to be as large, when measured by an instrument, as in the summer. This statement must not be confounded with the fact that under great falling temperatures the combined thermal stress of tension and that due to the positive bending moments in the base of the rail of the passing wheel effects may be and are often greater than those experienced in the summer, under the same locomotives. The thermal strains of tension reduce the factor of safety of the metal, while those of compression increase the looseness of the rails and the disturbance of the line. The tendency of the rails in summer, therefore, is to become slightly looser as continuous members, for the same standards of maintenance, and to creep most under the spikes.

The unit fiber strains under locomotives of two pairs of driving wheels are increased from 20 to 35% by the expended tractive effort, besides that due to the axle loads.

The superstructure of all steam roads is flexible, and is depressed temporarily from the "trackmen's surface" to the lower running surface in the "general depression" under the wheel base of the locomotives and cars. In the "general depression" there are specific deflections, in the rails under the wheels, of positive bending moments, constrained by negative bending moments in the wheel spacing. This constitutes the so-called "wave motion" in the superstructure. The ballast and subgrade are also loaded and partially unloaded, each wheel being distinctly felt in the subgrade, for 15 or 20 ft. in depth, in most places. Each type of locomotive in passing causes a characteristic "general depression" and bending moments, in loading the subgrade, and in extreme cases the rails and cross-ties show a slight movement with each train, and require attention to prevent creeping. Conditions in the permanent way which permit a depression of the rails of $\frac{1}{4}$ in. or more under the passing locomotives or cars, cause a tendency to creep. The outer rail on curves also tends to creep, with the traffic.

Decreasing the positive and increasing the negative bending moments in the rails, either by increased stiffness, better drained subgrade, a more favorable construction of the locomotive for the distribution of the total load and loading the rail, high standards of maintenance, with ample ballast, increase the combined stability between the passing locomotive and the permanent way, and check

creeping. This is proven, in the practice of the past decade, by the fast and heavy trains in daily service, compared with those on the former light rails. Dr. Dudley.

The 4-wheel "leading and guiding truck" for the engine, invented by the late John Bloomfield Jervis, Hon. M. Am. Soc. C. E., one of America's eminent civil engineers, has been the great factor in loading, not only the rails in the "general depression," but the subgrade.

Stephenson's unexcelled work was the combination for rapid steam generation for locomotives, while that of Jervis, his contemporary, the distribution of the load and ease of motion. The mechanical applications of both principles have exceeded the expectations of either, with less creeping of the rails than they experienced.

GEORGE TATNALL, M. AM. SOC. C. E. (by letter).—The author presents a very interesting series of observations on a troublesome subject, on which there are very few accurate data, and yet a great deal of speculation. Almost every maintenance-of-way engineer has his own pet theory of the cause of the creeping of rails, evolved from some personally observed and half-remembered facts. Mr. Tatnall.

The minute description of the methods of observation speaks well for the accuracy of the results obtained; nevertheless, there are some matters of importance decidedly lacking. The author quotes from Mr. Camp: "It is now generally conceded that the principal cause of creeping is the wave motion in the rails set up by moving trains." This coincides with the opinion expressed by himself, and yet no effort seems to have been made to ascertain the amplitude of this wave motion, in conjunction with the other observations.

The worst case of creeping that ever came under the writer's observation occurred on a wooden Howe truss bridge. This structure consisted of thirteen spans, each of about 150 ft., and a draw-span of 80 ft. clear, with 14 ft. between the trusses. The floor system was constructed of cross-beams of 8 by 16-in. yellow pine resting on the lower chord and spaced 16 in. apart, on which rested a longitudinal stringer of 8 by 12-in. yellow pine laid flat, on which the rail was spiked. This bridge carried a very heavy traffic, both in the number and weight of trains, and was considerably over-strained, so that constant attention was imperative and repeated repairs and renewals necessary. It has since been entirely replaced with a steel structure.

Upon this bridge, directly under the writer's eye, a creeping of rails of more than 1 in. with the passing of each train has been observed repeatedly. As the bridge was a single-track structure and the trains alternated in direction, the creeping caused by one train was balanced by the creeping in the other direction caused by the next. But this creeping did not occur under the driving-wheels, nor under the coach wheels, which has long been a bone of contention, but occurred entirely at the moment of the passage of the advance

Mr. Tatnall. upward wave, explained and described so clearly by Captain J. E. Howard, of the U. S. Arsenal at Watertown, Mass., in his reports on measurements of track deflections, and which occurs several feet in front of the engine. At the time of the observation of this excessive creeping, the trusses of several adjoining spans were three or four years old, and the deflection was beginning to be excessive; the deflection of the floor grades had increased and the rail had cut into the longitudinal stringer considerably, mashing the fibers, so that the amplitude of this advance wave was very great and very noticeable, and the force it exerted was sufficient, as the spikes had been driven down hard, to wear away the flanges of the rail under and on each side of the spike heads to an appreciable depth below the general plane of the flange surface. It should be stated that trains were not allowed to run at a speed of more than 15 miles per hour over this bridge.

Analyzing the data given by the author, it appears that at Bethayres and Pleasantville, where the creeping is the greatest, the tracks are in the one case on an embankment on low, swampy ground, and in the other on an embankment on the low-lying salt marshes of the coast; in both cases the compression of the embankment and underlying marsh by the heavy traffic would be relatively great and the wave motion of great amplitude.

The author quotes Mr. Camp as to the effect of temperature on the creeping of rails, and temperature is undoubtedly the second important factor. The creeping of the rails in hot weather toward the old-fashioned and now obsolete stub switch, is well known, as well as the creeping toward the free ends of the rails at a drawbridge; and, undoubtedly, many of the movements noted in these observations are the result of temperature changes, particularly the west-bound track at Hopewell and the north-bound track at North Wales and Gywnedd, as well as one or two other points, where whatever movement has been noticed is in a direction opposite to the traffic. It is to be regretted that the distance and direction to the nearest switch or crossing has not been noted as giving a partial clue to the influence of the temperature on the movement. When the difference between the summer length and the winter length of the two rails immediately adjoining a draw-span, in a bridge about 2 000 ft. long, amounts to 6 or 8 ft., it can readily be believed that the changes in temperature could cause greater backward and forward movements than the fluctuations shown in the tables.

In the writer's opinion, the wave motion, as influenced by the frequency and weight of the traffic, the compressibility of the roadbed, and the underlying natural surface, and the expansion of the rails, under increases of temperature toward an unstayed point in the track, are the only causes of creeping rails, and that such factors as curves, grades, speed of trains, nature of traffic, other than weight, etc., etc.,

are negligible quantities in the discussion. This opinion seems to be upheld by the data presented. Mr. Tatnall.

JOSEPH S. WARD, C. E. (by letter).—Although the writer has not been in very close touch with maintenance-of-way affairs in recent years, he was, prior to 1898, somewhat active in the investigation of this character of rail movement, and watched closely those points at which it was most marked on the Shamokin Division of the Philadelphia and Reading Railway, to which his experience and observation have, of late years, been almost entirely confined. Mr. Ward.

Since the renewal of interest excited by Mr. Wagner's paper, inquiry has been made of supervisors as to the rail movement, and, while their answers indicate comparative freedom from trouble by reason of such movement, the most noticeable cases are still existent at many of the places to which attention had been particularly directed while the writer was in charge of those tracks between 1890 and 1898.

The Shamokin Division, on account of its variable and excessive grades, long bridges and trestles, inclined planes, tortuous alignment, and character of traffic, offers a field for investigation which will, it is believed, if thoroughly explored, produce such data as may lead to the formulation of a more positive theory of the causes that induce rail creeping, and suggest the most effective means for its prevention.

While the writer does not feel competent, and it is not his intention, to attempt any scientific dissertation upon this phenomenon, he may be permitted expression of such opinion of some of the several theories advanced by the engineers who have discussed the paper as his experience seems to justify.

That of Mr. Lindenthal, by which he attributes "creeping" to the rolling friction of the wheels on the rails, and to the dynamic forces exerted thereby, is more pleasing and satisfactory than the explanation of Mr. Camp; for, upon more or less rigid substructures, where the "wave motion" is less than upon the comparatively elastic and compressible roadbeds, there have been many instances of excessive longitudinal track movement.

The writer feels quite satisfied of the correctness of Mr. Camp's statement, "that grades are a factor in rail creeping only as far as they influence the speed of trains," to which he would add, "and increase the push or dynamic forces exerted at the rail ends by reason of heavy braking and locked wheels."

He does not agree with Mr. Stevens' conclusion, "that track movement must precede the train," although "wave motion" may. The broken splices which are referred to, it is believed, may be correctly attributed to this undulatory movement, aggravated by the increased leverage imposed upon the unnecessarily long angle-plates used in recent years. While we have had numerous instances of joint parting and of rail separations even greater than those named by him, the bolts

Mr. Ward. were sheared off in all cases, and in none, within our knowledge, was there any evidence of cracked or broken plates.

Mr. Stevens has stated most succinctly the physical defects of tracks and roadbed which facilitate the creeping of rails, and proposes what is undoubtedly the most effective means for its prevention, *viz.*, ideal track.

The writer cannot reconcile with his experience the statement of Mr. McDonald, "that, no matter what the direction of the preponderance of traffic, the rail will always creep down grade." The evidence shows most positively that the movement is invariably with the current of trade, regardless of grades.

The observations of Messrs. McDonald and Tatnall as to the creeping on and in the vicinity of long bridges and trestles are corroborative of the writer's experience; but he cannot agree with the last-named gentleman, "that speed and curves are negligible quantities in this discussion."

With regard to the comments of Dr. Dudley, they being principally descriptive and explanatory of the findings of the author, the writer has nothing to say, except to call attention to the paragraph in which the proposition of Mr. Stevens is emphasized by the important statement that, "with a stable roadbed, good drainage, more rigid track, ample ballast and a high standard of maintenance, the creeping of rails may be practically stopped."

The writer has always found an excess tendency of the right rail to creep, and a greater movement of the high rail on curves.

The writer would offer the opinion that the causes to which the longitudinal track movement are due, named in the order of their effective influence, are:

First.—A preponderance of heavy traffic in the direction of the movement (regardless of grades);

Second.—The number and frequency of the dynamic forces exerted at the rail ends (largely dependent upon the physical conditions of track and roadbed);

Third.—The speed of trains;

Fourth.—The increased rolling friction on descending grades, due to excessive braking and locked wheels;

Fifth.—Inadequate track construction and its inferior maintenance (particularly as to tie-spacing, slot-spiking and insufficient ballast);

Sixth.—The compressibility of the roadbed and its underlying foundation;

Seventh.—The manner in which the weight is distributed on the rails;

Eighth.—Rail expansion and contraction due to variations of temperature.

The influences classed under Nos. 5 and 8 may probably be more

correctly stated as facilitating track creeping rather than causing it; and, while the effect of poor track is pronounced, the writer's experience does not warrant the attribution of any considerable movement to the compressibility of track substructures. In fact, in no instance has he observed any marked creeping upon the many miles of new grading for second track, built within the past few years, that was not, in his opinion, due to other local conditions. However, this may be accounted for by the more or less constant work expended upon such new track, immediately following, and for some time after, its construction, and the consequent enforced maintenance of good surface until the new roadbed had settled. Mr. Ward.

Beginning at the southern end of the Shamokin Division, the points at which appreciable rail movements now occur, as reported by the supervisors, in consecutive order, are:

First.—South-bound track, Tamaqua to East Mahanoy Junction, specifically between Tamaqua Tunnel and Mintzers;

Second.—South-bound track, ½ mile north of Gordon;

Third.—South-bound track at Locust Dale Junction;

Fourth.—South-bound track on trestle at Williamsport;

Fifth.—Single track on trestle and bridge at Rupert;

Sixth.—Single track on seven iron viaducts on the Catawissa Branch;

Seventh.—Single track at Ryan's Tunnel.

From this list it will be observed that, on this Division, there appear to be but thirteen points at which the creeping of rails is so noticeable as to attract the attention of supervisors, and that, of this number, nine are on or adjoining bridges or trestles of considerable length.

Inasmuch as the principal factor in the track disturbance upon these bridges is the same in all cases, attention will first be given to their consideration. The structure at Williamsport is a pile trestle, 2 100 ft. long, on a tangent extending some distance east and west of its ends. The bents are spaced 17 ft. between centers, each alternate bay being well braced longitudinally, and every bent is stayed laterally. The floor system is composed of four 9 by 16-in. stringers, upon which are superimposed, 7 by 9-in. ties, spaced 20 in., center to center, with 6 by 8-in. guard rails. The grade upon and west of the trestle is practically level, and east thereof, descends in the direction of traffic at the rate of 0.3 per cent. The track is laid with 80-lb. rails of standard section and angle splices having six $\frac{7}{8}$-in. bolts to the joint. The south-bound substructure and track were built in 1899, while the north-bound trestle, although thrice partially renewed since its construction in 1871, still rests on foundations of the original piling. Two iron bridges, on pile and masonry foundations, and with respective lengths of 80 and 100 ft., located close together, divide the trestle into

Mr. Ward

two parts of 700 and 1 400 ft., the shorter being at the eastern end. In the longer section there are four metal girder spans, each averaging 25 ft.

Since the commencement of the operation of the double track, five years ago, there has been a very heavy and increasing south-bound trade of loaded cars of freight and bituminous coal, the latter predominating, and the daily average movement probably approximates 1 000 cars.

The creeping of rails on this track, entirely in the direction of the trains, has averaged about 10 in. annually, and is unquestionably superinduced by the heavy south-bound trade, the variable character of the substructure and loose splices.

On account of the regular tie-spacing on the trestle and bridges, general slot-spiking is impracticable; it is rarely that a joint is full spiked through the slots; at many, all the slots come between the ties, and the restraining influence of the comparatively few spikes properly driven is absolutely inadequate. In fact, in most instances, their holding power has been destroyed by the shearing of the splice flanges (by creeping) back of the slots.

That there has been no movement at, nor displacement of, the frog, ½ mile north or west of the trestle, may be explained by the fact that the rails were laid during very warm weather, with close joints, which have now opened practically to the full limit allowed for expansion.

Further creeping will, the writer is satisfied, show a displacement of this frog, or will tear the rails apart. At this point, as well as at all others coming under the writer's notice, the movement indicated is that of the right rail, which is generally about double that of the other.

It is somewhat remarkable that the joints east of the trestle have closed for but a few hundred feet, although the explanation may lie in the fact that the rails were laid in the spring, and that the ties are not fully back-filled with ballast.

Between October 8th and November 4th, 1904, the Section Foreman has been compelled to cut out of the north-bound track 8 in. of rail, at a point 700 ft. west of the east end of the trestle, near the 100-ft. span bridge and beyond which there was no closing of joints.

This would indicate a movement in the direction of the traffic equivalent to 1½ in. annually. That it has been so much less on this track than on the south-bound is due, no doubt, to the shorter length of trestle over which the creeping took place, and because the north-bound traffic, while about equal to the south-bound, is made up almost entirely of empty cars.

On the Catawissa Branch, at Rupert, the North Branch of the Susquehanna River is crossed by an iron bridge, 1 400 ft. long, subdivided into nine spans of 150 ft. each. This bridge rests on masonry piers.

Mr. Ward.

At its north end is a trestle, 1 000 ft. long, of similar construction to that at Williamsport, except that its spans are but 12 ft., and bents of ordinary form, on stone foundations, take the place of piles. Both structures are on a level grade and on tangent, there being a 6° curve immediately north of the trestle. Approaching the trestle there is a long grade, averaging 0.6%, descending in the direction of the heavier traffic; and, on the right-hand rail, going south, there is, at the south end of the bridge, a single-point derailing switch protecting the Pennsylvania Railroad crossing. The north-bound movement on this branch is mostly of empty cars, while the traffic in the opposite direction is of about the same character and volume as the south-bound trade at Williamsport. Track conditions, as to rails, spiking, etc., are identical at the two points.

The rail movement at Rupert amounts annually to 18 in. for the right rail and 8 in. for the left, and is in the direction of the heavy trade.

In the writer's opinion, this creeping is largely attributable to the same causes as induce it at the Williamsport trestle, where the conditions of grade, etc., are similar; and the increased movement may be accounted for by the additional factors resultant from the descending grade north of the bridge, and the derailing switch at its south end, for which the right-hand rail serves as the stock rail, and is, therefore, free to run.

The writer is in receipt of a report that on October 8th, 1904, the right rails were torn apart at the north end of this trestle and the six $\frac{7}{8}$-in. bolts were sheared off at the open joint, the rails at which were said to have separated 26 in. Reference will be made later to an almost identical experience at the same place six or eight years ago.

The seven iron viaducts on the Catawissa Railroad, at Mainville, Mine Gap, Fishers, Long Hollow, Stranger Hollow, Ringtown and Dark Run, having respective lengths of 725, 757, 590, 746, 380, 1 077 and 570 ft., are, with the exception of that at Ringtown, which is practically level, all on grades equivalent to the average south-bound ascent of the branch between Catawissa and Lofty of 0.63 per cent. The construction of these bridges is identical in all instances, the open-deck spans having a length of 80 ft., and those over the towers being about 30 ft. long. The towers vary in height at the centers of the bridges from a minimum of 90 ft. to a maximum of 128 ft.

All these structures are on tangents, and at all of them, with the exception of the one at Dark Run, there are sharp curves at one or both ends, generally to the north. The spirals of these curves are on the viaducts, and, as the track movement is always to the south, its extent is, to a degree, influenced by this circumstance. The weight and pattern of rails are the same as at Williamsport and Rupert; complete slot spiking is equally impracticable; the same variation in the rigid-

Mr. Ward. ity of the substructures exists; the character and volume of the traffic in the direction of the creeping are practically identical at all nine bridges; and, as the rail movement, in every case, appears to be in direct proportion to the length of each structure (except where modified by such additional factors of disturbance as curvature and increased friction), one is forced to the conclusion that the same influences are at work at these several locations.

While those influences which are responsible for the creeping at the other points specifically mentioned as troublesome by the supervisors are similar in each instance, they differ from those to which this phenomenon has been attributed in the cases so far considered; at these four points the writer is not inclined to look upon track conditions or the compressibility of super- and sub-structures as important factors in the disturbance; and, is satisfied that volume and direction of traffic, speed of trains, increased rolling friction and curvature are alone contributory to the rail movement.

At Ryan's Tunnel the rails creep on the high side of an 11° 30′ curve, on a 0.6% grade at the foot of a long incline descending in the direction of the trade, at the rate of 59 ft. to the mile.

The movement reported between Mintzers and Tamaqua Tunnel, on the south-bound track, takes place on a long grade, averaging 1.2% which descends with the traffic, and upon which there are numerous curves. Between Mintzers and Tamaqua Tunnel, at the foot of this descent, and where the track disturbance is greatest, the grade drops off to 0.5%, the alignment is more tortuous, and there are two bridges with an aggregate length of 160 ft. upon which the splices are more or less loose.

North of Gordon, ½ mile, the next location named at which unusual creeping is noticed south-bound, the track in this direction is on a falling grade of 0.35%; the alignment shows a number of curves of small radii, with short intermediate tangents, and the lighter grade is at the foot of a very long descent averaging 1.2 per cent. Finally, on the south-bound track at Locust Dale Junction, the rail creeping is most marked on reversing 8° curves on a grade in the direction of traffic and with a rail movement of 2.1 per cent. Immediately north of this point there is a long tangent descending toward the point of maximum creeping at the rate of 2.54 per cent.

At all these four locations just described there is a heavy south-bound movement of loaded cars; the curvature is excessive and sharp; the grades, unusually heavy, change abruptly, in all instances, at or near the points of greatest disturbance; and, as the tracks of either 80- or 90-lb. rail are in good condition, well ballasted, slot spiked and on solid beds, the theory which the writer has advanced for the creepings seems to be well substantiated.

Although all the cases of most pronounced creeping, reported by

the supervisors, have now been discussed, such advice from other sources has been received as leads to the belief that this character of movement may be much more general than is supposed. One of these cases, in which a creeping on both tracks has been observed, occurs at or near Paxinos on the Shamokin, Sunbury and Lewisburg Railroad. Mr. Ward.

The Shamokin, Sunbury and Lewisburg Branch, between West Milton and Shamokin, is the only one of the lines comprising the Shamokin Division upon which the volume and character of north- and south-bound business is nearly equal, and it is to this fact that the rail creeping in both directions, just referred to, is probably due. On all other portions of the Division, and notably south of Shamokin, the preponderance of the loaded movement southward is excessive, reaching the maximum between East Mahanoy Junction and Tamaqua The trade in the opposite direction is made up of trains in which the empty cars are so greatly in excess of the loaded ones that their effect is not appreciable, as is demonstrated by the absence of any notable instances of creeping.

In conclusion, reference will now be made to the most pronounced instances of the phenomena under discussion, noticed between 1890 and 1898. These occurred on the Shamokin, Sunbury and Lewisburg Railroad, between Deiblers and Weigh Scale in 1890 and 1891; at Sunbury Bridge in 1891; at Rupert Bridge in 1897; at Mahanoy, North Ashland and Mahanoy Planes, 1890 to 1897; at Ashland Summit on the upper route and at Williamsport trestles. When the writer took charge in 1898, with the exception of the double track between Lewisburg and West Milton, that portion of the Division north of Shamokin and including the Catawissa Railroad was single track, laid with 70- and 68-lb. rails. The lighter rails were confined to the Shamokin, Sunbury and Lewisburg Railroad, and parts of the Catawissa Branch; they were but 24 ft. long and equipped with plain splices, or fish-plates, with four $\frac{3}{4}$-in. bolts to the joint. The 70-lb. rail was laid with light angle-plates having bolts of the same diameter and number.

On the Shamokin, Sunbury and Lewisburg Railroad, and particularly south of Deiblers, there was endless trouble; the track would creep alternately north or south as the traffic would fluctuate, frequently tearing apart at the joints and opening up from 2 to 4 in. It was almost a daily occurrence to receive notification of one or more such occurrences during the previous night, and, as it was impossible to secure the track, by reason of the absence of slots in either the splices or rails, serious accidents were averted only by the greatest vigilance. Upon the completion of the double track, the trouble was intensified by the diversion to this old 68-lb. track of all the south-bound business; joints parted by day and by night; the rails crept almost incredible distances daily; rail separations of from 4 to 11 in.

Mr. Ward. were frequent, and at least one wreck was occasioned by such a parting under the train. The presence in this single track, at the time the writer first knew it, of eleven hundred 4-ft. pieces of rail, or, as the trackmen designated them, "Dutchmen," in a distance of 25 miles, most of them in the 8 miles south of Deiblers, gives some idea of the conditions with which it was necessary to cope. These short rails were removed; no piece less than 12 ft. was permitted in the track; 68-lb. angle-splices were ordered and put on as fast as possible; joint ties were spaced; slots were spiked, and, with the continued improvement, creeping gradually diminished until it finally almost entirely disappeared, with the placement of the 80-lb. steel, standard 6-bolt joints and the slot-spiked ties.

During the operation of the single track between the points mentioned, the 0.5% grade ascending southward did not appear to influence the track movement, which would alternate with the trade, the preponderance of which was for a time in one direction and again in the other.

During this period, the track on one occasion parted, shearing off six $\frac{7}{8}$-in. bolts at a joint at the north end of Sunbury Bridge at the south end of the 10° 30' curve adjacent thereto.

The bridge was then, and is now, a single-track structure, 2 600 ft. long, on a level grade, and, as there has never been any further trouble there, it has always been difficult to account for the occurrence. It may be that the explanation lies in the fact that the rails on the curve and bridge were of the 76-lb. pattern, while the adjacent track, north and south, was of 68-lb. steel. The sharp curve at the north end may also have been a factor.

At Williamsport, at that time, there were about 8 000 or 9 000 ft. of trestle, or four times as great a length as now. It was old and of lighter construction than the standard of this date, and the rails on it were 70-lb., with angle-plates and four $\frac{7}{8}$-in. bolts at the joints. As it was possible to slot-spike occasionally, there was less trouble than on the Shamokin, Sunbury and Lewisburg Railroad; but the creeping was so great as to cause much anxiety, and, while no joints parted, no means of keeping the movement within safe limits could be devised. The slot-spiking at the joints was supplemented by check-straps and rail-slotting and spiking, but the flanges of plates were sheared off, the ties were split, the spikes pulled out, and the straps were torn loose. When for a space the rails were secured, the entire substructure would move, and the writer has frequently seen the trestle bents lean east or west, at angles of 75 or 80°, as the trade movement was greater in one direction or the other. Since these trestles have been filled and properly constructed, and heavier track laid, there has been absolutely no trouble on either the single or double track.

In 1897, the river bridge at Rupert was a wooden Howe truss, and,

while prior to that time there had been no evidence of serious creeping over it, under the light and approximately equal traffic in both directions, there was a sudden and radical change when, in that year, the bulk of the south-bound coal and freight was diverted at West Milton and sent over the Catawissa Branch. Within a day or two, the increased trade had carried the rails so far south that the joints parted at the north end of the trestle and opened nearly 3 ft. Mr. Ward.

After opening up the joints and cutting in rails of proper length to restore normal conditions, blocks were placed on the stringers, between the ties, to permit full slot-spiking at all joints; and, from that time there has been no trouble until the occurrence of October 8th, heretofore described. The wooden bridge was replaced by the iron one about four years ago, and the intermediate tie blocking was never applied thereto.

That the joints should part at Ashland Summit, which is on a 4° curve and from which the descending grades on either side are, north and south-bound, respectively, 1.2% and 3.0%, is not remarkable, and calls for no special explanation.

The most important illustrations of rail creeping against the grade, as exemplified by the inclined planes which for many years have formed such a special and important feature of the Shamokin Division, have been reserved for the last. These planes were four in number, all double-tracked and located at Mahanoy Plane, Gordon and Ashland, the last-named having been known as Big Mine Run Plane. That at Mahanoy Plane has a length of 2 450 ft., with a uniform grade of 17 per cent. The two planes at Gordon had respective lengths of 4 774 and 4 584 ft., with grades of 6.54, 11.98, 6.43 and 9 per cent. Big Mine Run Plane had a length of 1 200 ft., and a uniform grade of 22.5 per cent.

Over Mahanoy Plane is now hoisted the anthracite coal originating in the Shamokin and Gordon Districts, as well as that produced between Gordon and Gilberton. Prior to the abandonment of the Gordon and Big Mine Run Planes in 1899, all the coal mined in the Mahanoy Valley passed over this plane, while a large proportion of the east-bound freight and all the coal produced in the Gordon and Shamokin regions were handled *via* the Gordon Planes.

The annual tonnage between 1890 and 1898 over these two planes averaged for Mahanoy Plane, 3 600 000 tons, and for Gordon, 2 950 000 tons. At Big Mine Run, the plane was self-acting, the descending loaded cars on one track hoisting the empty cars on the other. Over it, in the writer's time, passed the product of but a single colliery, known as the "North Ashland," the tonnage of which approximated 150 000 annually.

At Mahanoy and Gordon Planes the loaded cars were hoisted up the plane alternately on each track, the descending movement being entirely of light cars. At Big Mine Run the loaded and empty movements were in reverse directions.

Mr. Ward. As said, these planes, with the exception of Mahanoy Plane, which is still active, were abandoned in 1899, and entirely dismantled.

At Mahanoy Plane, which is now equipped with 80-lb. rails and standard angle-plates, which are full-spiked at the slots, on properly spaced joint ties, the writer is informed that there is but little, if any trouble, experienced from rail creeping; what little there is, however, is always up the plane.

In 1890 the rails on all these plane tracks were of the 68-lb. or lighter patterns, with plain splices; there were no provisions made for the prevention of longitudinal movement: and the trouble from creeping was constant and serious. At Mahanoy and Gordon, the cutting out of 8 or 10 in. of rail at a time at the heads of planes was not infrequent; while, at Big Mine Run, daily attention to the stub switches at its foot was a necessity.

After the substitution of point switches for the stubs at this point, the rails would buckle about 150 ft. beyond the foot of the plane. With some degree of success, it was sought to lessen the creeping by replacing the fish-plates with angle-bars, spacing the joint ties and slot-spiking.

The ties then moved with the creeping to such an extent as to interfere with the bearings of the pulleys under the hoisting cables; and, compelled the placing of timber guard rails on the ties to correct this difficulty and to prevent additional wear on the ropes. It was not until the heavier rails were laid that the present satisfactory condition was attained. The rail movement on these planes is nearly equal on both rails, from which it would appear that the locomotives are responsible for, or influential in, the greater movement of the right rail at other points.

At all the locations named, there has always been more or less trouble, and, apart from them, rail creepings, while doubtless existent, are not perceptible in practice, give no annoyance and may be considered negligible.

The explanations offered for the instances of rail creeping herein described and drawn from the attendant conditions, seemingly justify the theories advanced on page 494.

Attention has been called to the influence of each one of the eight at one or more of the locations given, and the falsity of Mr. McDonald's proposition that "rails always creep down grade," has been proven. The actual results of these experiences and observations are, in any event, interesting, and the writer believes he can no more fittingly terminate this discussion than by their recapitulation, as follows:

First.—Rail creeping is invariably in the direction of the preponderance of traffic, regardless of grade.

Second.—It is greater on descending grades.

Third.—It is greatest on or adjacent to long bridges and trestles. Mr. Ward.

Fourth.—There is a more pronounced movement on the high rails of curves.

Fifth.—The locomotive is a factor in this increased movement.

Sixth.—The right rail creeps about 50% more than the left.

Seventh.—Creeping is increased by speed and curvature.

Eighth.—Compressibility of track and substructures, while active, is not an influential factor in creeping.

Ninth.—Track movement, if there be any, in a direction opposite to that of the volume of trade, is not appreciable.

Tenth.—Tie-plates are efficacious in its elimination.

Eleventh.—It is, on solid roadbeds, practically remediable.

Twelfth.—It may be checked on bridges or trestles by some such expedient as that resorted to at Rupert.

CHARLES RUFUS HARTE, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Wagner's paper is very interesting, and is a valuable contribution to the subject of rail creep. Mr. Harte.

It may be questioned, however, if the author has not neglected one factor of considerable importance, namely, expansion.

As pointed out by Mr. McDonald, expansion and contraction both tend to work the rails down grade, regardless of traffic direction. Track repairs in sags, particularly in the case of the average country electric railroad, are often much complicated by this tendency of the steel to come down the hill, and the writer has known of several instances where such movement has caused serious trouble.

Comparison of creepings on single- and double-track lines operating under identical conditions would at once show to what extent factors independent of traffic direction affect the movement, but such comparisons can be made but rarely.

A very short section of trackage of the New York, New Haven and Hartford Railroad at New Haven, Conn., does offer in part such opportunity, the double-tracked Hartford Division being paralleled by the single track of the Northampton Division. Here roadbed conditions are identical; but traffic on the Hartford Division is from four to five times as heavy as that on the Northampton Division.

In the hope of obtaining information of value, the writer, upon reading Mr. Wagner's paper, established two sets of ranges across the three tracks, and three other sets of ranges across the Hartford Division tracks only. The results are presented in Plate XXXIV, arranged in the form of Mr. Wagner's records for ease of comparison.

The "East Cut," in which the observations were taken, from Station 1 100 to Station 1 124, is the bed of the old New Haven and Northampton Canal, which at the latter point turns north, carrying the Northampton track.

From Station 1 100 to Station 1 119 the base of rail is about 1

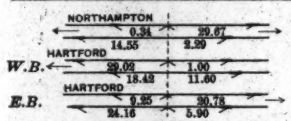
TABLE 5.—CREEPING RAILS, HARTFORD AND NORTHAMPTON DIVISIONS, N. Y., N. H. & H. R. R., "EAST CUT," NEW HAVEN, CONN.

Location.	Alignment.	Rails.	Ballast.	Joints.	Cut.	Traffic.						Remarks.
						Regular Daily Trains.						
Station 1 108.	P. C. of 7° 30' Curve Right, 200 ft. long.	N. Y., N. H. & H. Standard, 78-lb.	Gravel.	Suspended, slotted, and spiked on ties.	12 ft.	East Bound,	Freight.	21	7	Hartford and Northampton Divisions.		Alignment good. Subgrade, fine sand. Water table but little below bottom of tie.
Station 1 112.	Tangent.	"	"	"	10 ft.		Passenger.	37	7			As above.
						West Bound.	Freight.	16	7			
Station 1 125 + 90.	Center of 3° Curve Right, 200 ft. long.	N. Y., N. H. & H. Standard, 100-lb.	"	"	9 ft.		Passenger.	37	7	Hartford Division Only.		Alignment good. Water table well below bottom of tie. Subgrade, fine sand.
Station 1 146 + 75.	Center of 3°30' Curve Right, 600 ft. long.	"	"	"	15 ft.			Hartford Division.....	Northampton Division..			As above.
Station 1 162.	Center of 6° Curve Left, 1 000 ft. long.	"	"	"	3 ft.							As above.

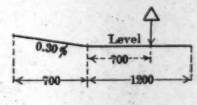
RECORD OF CREEPING RAILS. N.Y., N.H. & H.R.R., NEW HAVEN IMPRO

STA. 1108

LOCATION OF JOINTS



GRADE



NORTHAMPTON

N. Rail	Aug. 30	30.00		30.00
	Oct. 1	30.01		30.01
	Nov. 2	30.00		30.02
S. Rail	Jun. 3			
	Jun. 13	25.00		29.99
	Jul. 1	24.93	Reset	30.07
	Jul. 25	24.99		29.99
	Aug. 30	25.00		30.005
	Oct. 1	25.01		30.00
	Nov. 2	25.02		30.00

HARTFORD — W. BOUND

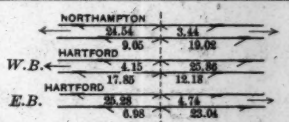
N. Rail	Aug. 30			30.00
	Oct. 1			30.005
	Nov. 2			30.01
S. Rail	Jun. 1			
	Jul. 22	30.00	Reset	30.02
	Aug. 30	30.00		30.04
	Oct. 1	29.99		30.04
	Nov. 2	29.99		30.03

HARTFORD — E. BOUND

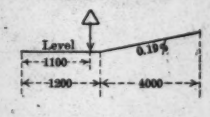
N. Rail	Aug. 30	30.00		30.00
	Oct. 1	30.00		30.00
	Nov. 2	29.99		29.99
S. Rail	Jun. 3			
	Jun. 13	29.99		29.99
	Jul. 1	29.96		30.04
	Jul. 25	30.00		29.99
	Aug. 30	30.01	30.00	
	Oct. 1	30.00	30.00	
	Nov. 2	30.00	30.01	

STA. 1112

LOCATION OF JOINTS



GRADE



NORTHAMPTON

N. Rail	Jul. 25	30.01		30.01
	Aug. 29	30.025		30.015
	Oct. 1	30.01		30.015
	Nov. 2	30.02		30.02
S. Rail	Jun. 1			
	Jun. 13	29.99		29.99
	Jul. 25	30.00		30.01
	Aug. 29	30.00		30.01
	Oct. 1	30.00		30.01
	Nov. 2	30.02		30.02

HARTFORD — W. BOUND

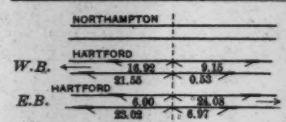
N. Rail	Jul. 25			
	Aug. 29	30.01		30.00
	Oct. 1	30.01		30.00
	Nov. 2	30.01		30.00
S. Rail	Jun. 15	29.61		30.30
	Jul. 25	30.01		29.99
	Aug. 29	30.02	Reset	29.99
	Oct. 1	29.96		29.93
	Nov. 2	29.97		29.97

HARTFORD — E. BOUND

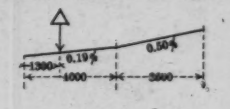
N. Rail	Jul. 25	30.00	30.015	
	Aug. 30	29.98	30.015	
	Oct. 1	29.975	30.01	
	Nov. 2	29.99	29.99	
S. Rail	Jun. 1			
	Jun. 13	29.99		29.99
	Jul. 25	30.00		30.00
	Aug. 30	30.00	30.00	
	Oct. 1	30.01	30.00	
	Nov. 2	30.01	30.00	

STA. 1125+90

LOCATION OF JOINTS



GRADE



NORTHAMPTON

N. Rail				
S. Rail				

HARTFORD — W. BOUND

N. Rail	Aug. 30	30.00		30.00
	Oct. 1	30.00		30.00
	Nov. 2	30.02		29.99
S. Rail	Jun. 3			
	Jun. 13	29.98		29.98
	Jul. 23	30.01		29.96
	Jul. 29	29.99	Reset	30.00
	Aug. 30	30.005		30.005
	Oct. 1	30.01		30.001
	Nov. 2	30.02		29.99

HARTFORD — E. BOUND

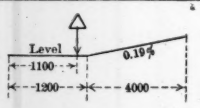
N. Rail	Aug. 30	30.00		30.00
	Oct. 1	30.01		30.05
	Nov. 2	30.02		30.01
S. Rail	Jun. 3			
	Jun. 13	29.98		29.99
	Jul. 5	29.96		30.03
	Jul. 23	30.00		29.99
	Jul. 30			
	Aug. 30	30.00		30.00
	Oct. 1	30.01		30.00
	Nov. 2	30.00		30.00

PLATE XXXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIII, No. 987.
HARTE ON
CREEPING OF RAILS.

RECORD OF CREEPING RAILS. N.Y., N.H. & H.R.R., NEW HAVEN IMPROVEMENT.

STA. 1112

GRADE



...TON

N. Rail		30.01
		30.015
		30.015
		30.02
S. Rail		29.99
		30.01
		30.01
		30.01
		30.02

...RD — W. BOUND

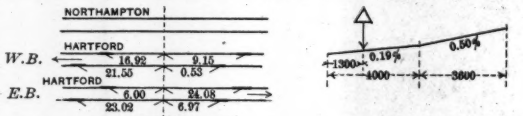
N. Rail		30.00
		30.00
		30.00
S. Rail		30.30
	Reset	29.99
		29.95
		29.67

...RD — E. BOUND

N. Rail		30.015
		30.015
		30.01
		29.99
S. Rail		29.99
		30.00
		30.00
		30.00

STA. 1125+90

LOCATION OF JOINTS — GRADE



NORTHAMPTON

N. Rail		
S. Rail		

HARTFORD — W. BOUND

N. Rail	Aug. 30	30.00
	Oct. 1	30.00
	Nov. 2	30.02
		30.00
		30.00
		29.99
S. Rail	Jun. 3	
	Jun. 13	29.98
	Jul. 23	30.01
	Jul. 30	29.99
	Aug. 30	30.005
	Oct. 1	30.01
	Nov. 2	30.02
	Reset	
		29.98
		29.96
		30.00
		30.005
		30.001
		29.99

HARTFORD — E. BOUND

N. Rail	Aug. 30	30.00
	Oct. 1	30.01
	Nov. 2	30.02
		30.00
		30.05
		30.01
S. Rail	Jun. 3	
	Jun. 13	29.98
	Jul. 5	29.96
	Jul. 23	30.00
	Jul. 30	
	Aug. 30	30.00
	Oct. 1	30.01
	Nov. 2	30.00
		29.99
		30.05
		29.99
		30.00
		30.00
		30.00

STA. 1146+75

LOCATION OF JOINTS — GRADE

HARTFORD — W. BOUND

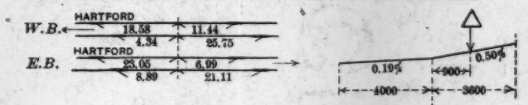
N Rail	Jun. 1	
	Jun. 13	29.99
	Jul. 1	30.04
	Jul. 23	30.00
	Jul. 30	
	Aug. 30	29.99
	Oct. 1	30.01
	Nov. 2	
		30.00
		29.98
		30.01
		30.00
		30.00
		29.99
S. Rail	Aug. 30	30.00
	Oct. 1	30.00
	Nov. 2	30.01
		30.00
		30.00
		30.00

HARTFORD — E. BOUND

N. Rail	Jun. 1	
	Jun. 13	29.99
	Jul. 1	30.02
	Jul. 23	29.98
	Jul. 30	
	Aug. 30	30.01
	Oct. 1	29.99
	Nov. 2	30.00
		29.99
		29.99
		29.99
		30.00
		30.01
S. Rail	Aug. 30	30.00
	Oct. 1	29.98
	Nov. 2	29.99
		30.00
		30.01
		30.015

STA. 1162

LOCATION OF JOINTS — GRADE



HARTFORD — W. BOUND

N. Rail	Jun. 1	30.00
	Jun. 13	30.00
	Jul. 1	30.05
	Jul. 23	30.00
	Aug. 30	30.00
	Oct. 3	30.005
	Nov. 2	30.01
		30.06
		29.99
		29.94
		30.01
	Reset	
		30.00
		29.995
		30.00
S. Rail	Aug. 30	30.00
	Oct. 3	30.00
	Nov. 2	29.99
		30.06
		30.00
		29.99

HARTFORD — E. BOUND

N. Rail	Jun. 1	30.00
	Jun. 13	30.00
	Jul. 1	30.045
	Jul. 23	30.01
	Aug. 30	30.00
	Oct. 3	30.01
	Nov. 2	30.02
		30.00
		29.99
		29.995
		30.00
		30.00
		30.00
S. Rail	Aug. 30	
	Oct. 3	30.00
	Nov. 2	30.00
		30.00
		30.00

NOTES:-

Sta. 1108 P.C. of 7°30′ Curve Right;
Curve 200′ long.
Sta. 1112 Tangent
Sta. 1125+90 Center of 8° Curve Right;
Curve 300′ long.
Sta. 1146+65 Center 3°30′ Curve Right;
Curve 600′ long.
Sta. 1162 Center of 6° Curve Left;
Curve 1000′ long.

ft. below extreme high water in New Haven Harbor; the combination of a heavy rain on a flood tide often floods this portion of the cut to, or even above, the top of rail, and at all times the sand of the subgrade is full of water. Owing to the extremely limited headroom, the tracks, up to Station 1 125, are laid with the New York, New Haven and Hartford Standard 78-lb. rail, the suspended joint being notched and spiked to the ties. Mr. Harte.

From Station 1 119 the track rises to the east, changing at Station 1 125 from 78-lb. to 100-lb. rails (also New York, New Haven and Hartford Standard), and having the same type of joint as the lighter rail. This section is well drained. The entire length of the cut is in gravel ballast. Owing to the imminence of extensive improvements, this trackage, while in good condition, has not been brought up to the high standard of the main lines on either side.

The data are too restricted, both in quantity and in extent of field, to warrant any deductions; they are presented in order that such actual measurements as have been made may be together. Attention, however, is called to the marked differences between the results on the Northampton tracks at Stations 1 108 and 1 112, only 400 ft. apart.

It should be noted that the Passenger Station is at Station 1 185, and that practically all trains are accelerating from that point to well beyond Station 1 162.

There are, in addition to the regular trains, from 50 to 200 extra freights daily over the Hartford Division tracks. The daily traffic through the cut averages 300 trains.

SAMUEL TOBIAS WAGNER, M. AM. SOC. C. E. (by letter).—The writer has been somewhat disappointed, but not altogether surprised, that the discussion has brought forth so few similar observations by other railway systems in order to ascertain in a fairly scientific manner the conditions which cause the phenomenon of creeping rails. Mr. Wagner.

The discussion of Dr. Dudley would seem to indicate that his inquiries in behalf of the International Railway Congress brought forth but few similar observations, and it is unfortunate that the limited space allowed in his report will prevent their publication.

It would, therefore, seem to the writer that Theodore Voorhees, M. Am. Soc. C. E., First Vice-President, and William Hunter, M. Am. Soc. C. E., Chief Engineer of the Philadelphia and Reading Railway Company, are to be congratulated for their liberality in allowing such a full set of observations to be made.

In the discussion of the methods used Mr. Tatnall regrets that no effort was made to ascertain the amplitude of the wave motion in conjunction with the other observations. The details, as to the data to be collected, were left entirely to the writer, who is free to confess that he did not think of making such observations. At the present time, however, he is at a loss to know how such observations could have

Mr. Wagner. been made without the expenditure of very much more time and money than was allowed. As it was, the time occupied was considerable, on account of the distance between the several stations, and it required strenuous work to accomplish what was done in a reasonable time. The writer does not think that such observations would have been of much value unless they could have been taken regularly at each station occupied, which would have consumed so much time as to make it practically prohibitive. As it is, especially in view of the discussion which has been brought out, it is to be regretted that more observations were not taken on curves, and especially on bridges and trestles, in which case, the results would have been much more complete. Many of the instances cited in the discussion were either on bridges or trestles, and most of them are of great interest.

After selecting the various branches of the system because they represented different types of grades, speed and traffic, the detailed locations of the points of observation were generally selected with the desire to note points at the top and bottom of the maximum grades, in order to ascertain the effect of the grades upon the creeping under each kind of speed and traffic. The discussion of Mr. Ward with special reference to the motion on the Mahanoy Plane, which is an extreme exaggeration upon the effect of grades, seems to aid materially in the confirmation of the fact that the greatest motion is in the direction of the heaviest traffic, irrespective of the grades. This fact seems to be agreed to by Mr. Lindenthal, Mr. Camp and Mr. Stevens. A majority of the writer's observations also tends to confirm it.

Mr. Ward calls attention to a point on the Shamokin Division, south of Locust Dale Junction, where creeping is especially noticed by the section foreman. This point is on some sharp curves, a few hundred feet south of the observation station of "Locust Dale Junction," and where no special creeping was noted. After a number of observations had been made at this point, the writer's attention was called to this fact by the section foreman. It is illustrative of the extreme local action of creeping in many places.

The question of the relative movement of the right and left rails has been brought up very strongly by Mr. Ward, who comes to the conclusion that the greatest motion is generally in the right rail, and that it creeps 50% more than the left. The observations in the paper are not conclusive on this point, although at the points of greatest movement, *viz.*, "Bethayres" on the New York Branch, and "Broad Mountain" and "Head of Grade" on the Frackville Branch, a greater movement of the right rail is shown. On the other hand, at Pleasantville, on the Atlantic City Railroad, the left rail moved more than the right. As all these observations were on double track, and, therefore, on the outside rail, the possibility of the greater movement being caused by the ties being less firmly held on the outside of the

roadbed, arises, and is interesting. Wherever there is much move- Mr. Wagner.
ment of the rail, it is always indicated to a greater or less extent by the "bunching" of the ties, Mr. Camp calls attention* also to the fact that:

"Track inspection apparatus which measures rail deflection, show that almost invariably the rails on the outside of double track are subject to greater average deflection per mile than the inner rails."

Mr. Ward's record of the creeping of the right rail in excess of the left on the Williamsport trestle is interesting, and seems to have no explanation based on any of the theories just mentioned. At the bridge at Rupert, where the same thing occurred, it can be readily accounted for by the presence of the derailing switch.

That a very large proportion of the movements recorded in the observations of the paper, especially where they do not indicate a consecutive continuous movement, are due to expansion and contraction, s the opinion of the writer, and this view seems to be held by most of those who have discussed the paper. The measurements which were made longitudinally across the rail joints were for the purpose of determining this question, and, in the majority of the cases, show that where these slight differences occur there is some movement in the joints. Even where the splice-plates were bolted so tightly that expansion and contraction could not take place at the joint observed, it probably did some little distance away. The variations of temperature between the several observations were quite great in many cases, and would readily account for the slight movements recorded.

The observations made by Mr. Harte are interesting, and afford easy comparison with the writer's tables. The fact of being able to make a comparison between single- and double-track lines is unusual, and seems to confirm the fact that movement on single-track lines is not likely to be troublesome. Mr. Harte's observations also show very clearly that the movement is in the direction of the traffic, independent of the grades, although the grades in his case are light. The amount of the movement on the double-track line is quite noticeable, but is probably caused by two conditions: First, the character of the subgrade at Stations 1 108 and 1 112, and, second, the trackage, while in good condition, is not up to the standard at other points of the line. While it is probably unwise to attempt to draw too many conclusions from a few observations, the writer feels that the difference in the movements between that at Stations 1 108 and 1 112, where 78-lb. rails were used, and that at Stations 1 125 + 90, 1 146 + 75 and 1 162, where 100-lb. rails were in the track, may in some measure be due to the weight of the rails.

The discussion has brought out very clearly the fact that the wave motion of the track, before and under the passing train, is not a

* "Notes on Track Construction and Maintenance," p. 588.

Mr. Wagner. universally accepted theory of the creeping of rails. Mr. Lindenthal makes a very strong argument in favor of the creeping being caused by the rolling friction of the wheels on the rails and the dynamic effect of the blow of the rapidly revolving wheels on the rail caused by the irregularities of the track, and states that if a track were built which could not be subject to wave motion that there would still be creeping of the rails, although at a diminished rate. Mr. Ward comes to practically the same conclusion when in his summary he states: "Compressibility of track and substructures, while active, is not an influential factor in creeping." And at another point he says:

"The writer is not inclined to look upon track conditions or the compressibility of super- or sub-structures as important factors in the disturbance; and, is satisfied that volume and direction of traffic, speed of trains, increased rolling friction and curvature are alone contributory to the rail movement."

The wave theory is generally held by the others as the primary cause, and the writer believes it to be the controlling factor, from a practical standpoint, although the arguments advanced by Mr. Lindenthal are certainly very strong, especially when applied to locations where creeping occurs, and where the wave motion cannot be very great.

If we conceive a train passing over a line of rails free to move vertically, so that vertical wave motion is possible, it is easy to believe that were there no rolling friction or other dynamic action tending to move the rails longitudinally, that after the passage of the train the rail would come to rest in its original position, or even if it had moved longitudinally, it might settle into a position which would be opposite to the direction of the passing train. All the evidence, however, goes to prove that the resultant motion is in the direction of the train, and that its extent depends largely upon the velocity. Even with the action of gravity against it, as in the case of the Mahanoy Plane, mentioned by Mr. Ward, with its grade of 17%, the resultant movement is up the grade in the direction of the heaviest traffic. It would seem, therefore, that with the rail in a condition where it is free to move, its movement is directed by the rolling friction or other dynamic forces present; but, eliminate the possibility of wave motion, and the creeping is practically negligible, while, theoretically, it is present to a small extent; so that while it is probable that the directing action is the friction, etc., they are of no practical moment if the condition of the roadbed, the weight of the rail and the character of the fastening are such as to prevent an abnormal wave motion of the rail under the passing train.

It would seem that both Mr, Lindenthal and Mr. Ward base a considerable part of their argument on the movement of the rail on bridges and similar structures, and, at first sight, it would seem that when the rail does move, under such circumstances, it must be due to the

causes mentioned by them. The writer, however, feels that, in considering such cases, due allowance must be made for the fact that, generally, the rail is in a condition in which it is much more free to move longitudinally than when on the ordinary roadbed, on account of the difficulty of spiking through the slots in the splice-bars. Neither Mr. Ward nor Mr. Lindenthal gives any accurate figures for the creeping on these structures of the more substantial sort, but it is evident that the rails do move materially. In Mr. Ward's cases, special reference is made to the Catawissa trestles, which are very substantial steel-deck structures of recent construction and the best modern design, with the girders spaced at 9 ft. centers and with a standard wooden floor, and the writer feels that under these circumstances there must necessarily be sufficient wave motion caused by the deflection of the ties to amplify the movement in the direction of the traffic, as explained above. Mr. Wagner.

Mr. Lindenthal, Mr. Stevens and Mr. Ward call special attention to the increased creeping of the outer rail on curves. That this is caused by the friction of the wheels against the outer rail is evident by the examination of the condition of any such rail in the track, and needs no further comment.

The troubles mentioned by Mr. Ward on the Shamokin, Sunbury and Lewisburg Railroad are extremely interesting, where, with the light rail, great numbers of "Dutchmen" and imperfect track, the rail movement was dangerous. This trouble was eliminated by the introduction of heavier rails and the improvement of the track as a whole. All the evidence seems to point to the fact that as heavier rails, better drainage of the roadbed, better ballast and better care of the track are introduced, the phenonenon of creeping is reduced so much that it attracts but little notice. The writer's attention was especially called to this fact, on those divisions of the system which had the highest standard of excellence in the roadbed, that very little information was to be obtained regarding creeping, in spite of the fact that on nearly all such divisions the traffic was the heaviest on the system, and that on all of them the speed was the highest. And, as all such divisions were double-tracked, these conditions should have produced creeping of a noticeable amount, as it appears that where creeping does occur it is practically irresistible.

MEMOIRS OF DECEASED MEMBERS.

GEORGE CURTIS TINGLEY, M. Am. Soc. C. E.*

DIED APRIL 30TH, 1904.

George Curtis Tingley was born in Windham, Connecticut, on January 28th, 1831, and died 73 years later, April 30th, 1904, at his home in Providence, Rhode Island.

His early education was obtained at the South Windham Academy, and in 1848 he entered Trinity College, Hartford, graduating with the class of 1852. With the exception of a few years immediately after completing his education, during which time he was engaged partly in teaching school, and partly in mercantile pursuits, his entire life was devoted to his profession of engineering.

He was Assistant Engineer on the location and construction of the Hartford, Providence and Fishkill Railroad, between Hartford and Providence (now a part of the New York, New Haven and Hartford Railway System).

About 1860 he entered the office of the late Samuel B. Cushing, Sr., M. Am. Soc. C. E., who was at that time the leading engineer of Rhode Island, and his connection with this office was maintained, in one capacity or another, for a period of thirty-three years.

For the first few years of his connection with the office, the firm name was Cushing and DeWitt, and upon the retirement of Mr. DeWitt, Mr. Tingley succeeded to his branch of the business, which carried with it the construction of the street railroad, at that time in its early stages of operation, and all the work then being done for the great manufacturing firm of A. and W. Sprague. About 1871 the firm of Cushing and Company was formed, which consisted of S. B. Cushing, the elder, his son, S. B. Cushing, Jr., George C. Tingley, and Col. J. Albert Monroe, all at that time or later Members of the American Society of Civil Engineers. This firm was dissolved by the retirement of Col. Monroe and the subsequent death of the elder Mr. Cushing in 1873, after which time it became S. B. Cushing and Company, composed of the younger Mr. Cushing and Mr. Tingley, and so continued until the death of Mr. Cushing in 1888, when Mr. Tingley conducted the business alone until his retirement from private practice in 1892.

* Memoir prepared by John W. Ellis, W. H. G. Temple, Herbert E. Sherman and Richard H. Tingley, Members, Am. Soc. C. E.

In 1892 Mr. Tingley was chosen Chief Engineer of the Union Railroad (now the Rhode Island Company), of Providence, and retained that position until his retirement from actual business in 1901. Mr. Tingley's connection with the engineering of the Union Railroad was a long and active one, covering a period of more than forty years from the time of the establishment of the first horse car lines in Providence to the date of his retirement from practice.

During the period of Mr. Tingley's long and successful private practice of engineering, many important works were designed and constructed by him, his chief attention being given to heavy foundation and bridge work, and to railway engineering. From his office have graduated a long list of engineers, Elmer L. Corthell, Desmond FitzGerald, Henry W. Parkhurst, Herbert E. Sherman, William H. G. Temple, Richard H. Tingley, all Members of the American Society of Civil Engineers, besides a large number of others, successful engineers, not members of this Society.

Mr. Tingley was married twice. First, in 1853, to Georgianna Sage, of Hartford. Four children survive this marriage: Pauline E. Tingley, of Providence, Richard H. Tingley, of New York, Mrs. Wm. D. Livermore, of Lawrence, Massachusetts, and Ernest DeW. Tingley, of New Haven, Connecticut. His second marriage was with Elizabeth Vaughan Polleys. Mr. Tingley had been a widower for five years prior to his death.

Mr. Tingley's entire life from early manhood to ripe old age was crowded with active responsibilities fully equal to those which fall to the average man. These he met and discharged with the highest ability and integrity, yet in the midst of these crowded duties he still found time for the exercise of a benevolence and charity rarely equalled in its usefulness.

Gentle, noble and kind, he has left the most endearing memories with all who knew him.

Mr. Tingley was elected a Member of the American Society of Civil Engineers on September 6th, 1871.

TRANSACTIONS

OF THE

American Society of Civil Engineers.

INDEX.

VOLUME LIII.

DECEMBER, 1904.

SUBJECT INDEX, PAGE 513.
AUTHOR INDEX, PAGE 517.

Titles of papers are in quotation marks when given with the author's name.

VOLUME LIII.

SUBJECT INDEX.

ADDRESSES.

Presidential Address at the Annual Convention at St. Louis, Mo., October 3d, 1904. Charles Hermany. 452.

BRIDGES.

"General Methods for the Calculation of Statically Indeterminate —, as Used in the Check Calculations of Designs for the Manhattan Bridge and the Blackwell's Island Bridge, New York." Frank H. Cilley. 413.

BUILDINGS.

"The Collapse of a Building during Construction." H. de B. Parsons. 1. Discussion: Nathaniel Roberts, C. J. Tilden, F. T. Llewellyn, James P. Whiskeman, Guy B. Waite, C. C. Schneider, Oscar Lowinson, H. P. Macdonald, J. H. O'Brien and George A. Just, 10.

CANALS.

"The Gatun Dam." C. D. Ward. (With Discussion.) 36.

CAST-IRON.

The use of — in building construction. 1.

COLUMNS.

"The Collapse of a Building during Construction." H. de B. Parsons. (With Discussion.) 1.

DAMS.

"Lake Cheesman Dam and Reservoir." Charles L. Harrison and Silas H. Woodard. 89. Discussion: Joseph P. Frizell, Burr Bassell, F. B. Maltby, E. Sherman Gould, E. W. Harrison, Frank C. Horn, Charles S. Gowen, Edward Wegmann, J. Waldo Smith, E. Kuichling, R. Shirreffs, George Y. Wisner, Edwin Duryea, Jr. and G. S. Williams, 133.

"The Gatun Dam." C. D. Ward. 36. Discussion: Charles L. Harrison, 41.

The Six-Mile Creek Dam. 183.

EARTH PRESSURES.

"Lateral — and Related Phenomena." E. P. Goodrich. 272. Discussion: M. F. Bonzano, V. H. Hewes, H. F. Dunham, Robert Brewster Stanton and Richard Lamb, 305.

EARTHWORK.

The compressibility of soils. 272.

FILTRATION.

"— for Public Water Supplies, with Especial Reference to the Double — Plant at Bremen, Germany." Eugen Goetze. 210. Discussion: W. Kiersted, Howard W. Underwood, William B. Fuller, George A. Soper, T. H. Wiggin, Morris Knowles, Allen Hazen, John H. Gregory, William R. Copeland, L. J. Le Conte, J. P. A. Maignen, George W. Fuller, William B. Bryan and Rudolph Hering, 228.

"On Sedimentation." Allen Hazen. (With Discussion.) 45.

LANDSLIDES.

"A Phenomenal Land Slide." (At Portland, Ore.) D. D. Clarke. 322. Discussion: George L. Dillman, Arthur L. Adams and James D. Schuyler, 398.

Cause of —. 307.

LAW.

Building laws. 1.

MEMOIRS OF DECEASED MEMBERS.

Tingley, George Curtis. 510.

RAILS.

"Some Notes on the Creeping of —." Samuel Tobias Wagner. 466. Discussion: G. Lindenthal, W. M. Camp, F. S. Stevens, Hunter McDonald, P. H. Dudley, George Tatnall, Joseph S. Ward and Charles Rufus Harte, 479.

RAINFALL.

— of the South Platte River Basin. 90, 153.

RESERVOIRS.

"A Phenomenal Land Slide." (At Portland, Ore.) D. D. Clarke. (With Discussion.) 322.

"Lake Cheesman Dam and Reservoir." Charles L. Harrison and Silas H. Woodard. (With Discussion.) 89.

RETAINING WALLS.

"Lateral Earth Pressures and Related Phenomena." E. P. Goodrich. (With Discussion.) 272.

SEDIMENTATION.

"On —." Allen Hazen. 45. Discussion: Galen W. Pearsons, Robert Spurr Weston and George W. Fuller, 72.

TRACK.

"Some Notes on the Creeping of Rails." Samuel Tobias Wagner. (With Discussion.) 466.

WATER.

"Filtration for Public — Supplies, with Especia Reference to the Double Filtration Plant at Bremen, Germany." Eugen Goetze. (With Discussion.) 210.

AUTHOR INDEX.

ADAMS, ARTHUR L.
A phenomenal land slide. 400.

BASSELL, BURR.
Lake Cheesman Dam and Reservoir. 137.

BONZANO, M. F.
Lateral earth pressures. 305.

BRYAN, WILLIAM B.
Filtration of water. 265.

CAMP, W. M.
The creeping of rails. 481.

CILLEY, FRANK H.
"General Methods for the Calculation of Statically Indeterminate Bridges, as Used in the Check Calculations of Designs for the Manhattan Bridge and the Blackwell's Island Bridge, New York." 413.

CLARKE, D. D.
"A Phenomenal Land Slide." 322.

COPELAND, WILLIAM R.
Filtration of water. 254.

DILLMAN, GEORGE L.
A phenomenal land slide. 398.

DUDLEY, P. H.
The creeping of rails. 488.

DUNHAM, H. F.
Lateral earth pressures. 306.

DURYEA, EDWIN, Jr.
Lake Cheesman Dam and Reservoir. 172.

FRIZELL, JOSEPH P.
Lake Cheesman Dam and Reservoir. 133.

FULLER, GEORGE W.
Filtration of water. 263.
On sedimentation. 77.

FULLER, WILLIAM B.
Filtration of water. 231.

GOETZE, EUGEN.
"Filtration for Public Water Supplies, with Especial Reference to the Double Filtration Plant at Bremen, Germany." 210.
GOODRICH, E. P.
"Lateral Earth Pressures and Related Phenomena." 272.
GOULD, E. SHERMAN.
Lake Cheesman Dam and Reservoir. 139.
GOWEN, CHARLES S.
Lake Cheesman Dam and Reservoir. 147.
GREGORY, JOHN H.
Filtration of water. 250.
HARRISON, CHARLES L.
"Lake Cheesman Dam and Reservoir." 89.
The Gatun Dam. 41.
HARRISON, E. W.
Lake Cheesman Dam and Reservoir. 144.
HARTE, CHARLES RUFUS.
The creeping of rails. 503.
HAZEN, ALLEN.
Filtration of water. 245.
"On Sedimentation." 45.
HERING, RUDOLPH.
Filtration of water. 269.
HERMANY, CHARLES.
Presidential address at the Annual Convention at St. Louis, Mo., October 3d, 1904. 452.
HEWES, V. H.
Lateral earth pressures. 306.
HORN, FRANK W.
Lake Cheesman Dam and Reservoir. 146.
JUST, GEORGE A.
Collapse of a building during construction. 32.
KIERSTED, W.
Filtration of water. 228.
KNOWLES, MORRIS.
Filtration of water. 238.
KUICHLING, E.
Lake Cheesman Dam and Reservoir. 153.
LAMB, RICHARD.
Lateral earth pressures. 314.
LE CONTE, L. J.
Filtration of water. 256.

LINDENTHAL, GUSTAV.
The creeping of rails. 479.
LLEWELLYN, F. T.
Collapse of a building during construction. 15.
LOWINSON, OSCAR.
Collapse of a building during construction. 26.
MACDONALD, H. P.
Collapse of a building during construction. 30.
McDONALD, HUNTER.
The creeping of rails. 487.
MAIGNEN, J. P. A.
Filtration of water. 257.
MALTBY, F. B.
Lake Cheesman Dam and Reservoir. 138.
O'BRIEN, J. H.
Collapse of a building during construction. 31.
PARSONS, H. DE B.
"The Collapse of a Building during Construction." 1.
PEARSONS, GALEN W.
On sedimentation. 72.
ROBERTS, NATHANIEL.
Collapse of a building during construction. 10.
SCHNEIDER, C. C.
Collapse of a building during construction. 24.
SCHUYLER, JAMES D.
A phenomenal land slide. 405.
SHIRREFFS, R.
Lake Cheesman Dam and Reservoir. 155.
SMITH, J. WALDO.
Lake Cheesman Dam and Reservoir. 152.
SOPER, GEORGE A.
Filtration of water. 232.
STANTON, ROBERT BREWSTER.
Lateral earth pressures. 307.
STEVENS, F. S.
The creeping of rails. 485.
TATNALL, GEORGE.
The creeping of rails. 491.
TILDEN, C. J.
Collapse of a building during construction. 13.
TINGLEY, GEORGE CURTIS.
Memoir of. 510.

UNDERWOOD, HOWARD W.
Filtration of water. 229.
WAGNER, SAMUEL TOBIAS.
"Some Notes on the Creeping of Rails." 466.
WAITE, GUY B.
Collapse of a building during construction. 21.
WARD, C. D.
"The Gatun Dam." 36.
WARD, JOSEPH S.
The creeping of rails. 493.
WEGMANN, EDWARD.
Lake Cheesman Dam and Reservoir. 150.
WESTON, ROBERT SPURR.
On sedimentation. 74.
WHISKEMAN, JAMES P.
Collapse of a building during construction. 17.
WIGGIN, T. H.
Filtration of water. 235.
WILLIAMS, G. S.
Lake Cheesman Dam and Reservoir. 182.
WISNER, GEORGE Y.
Lake Cheesman Dam and Reservoir. 170.
WOODARD, SILAS H.
"Lake Cheesman Dam and Reservoir." 89.